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CALTRAIN SAN FRANCISCO
DOWNTOWN EXTENSION PROJECT
CONCEPTUAL DESIGN AND DRAFT EIS/EIR

Volume I - Main Report

Geotechnical Site Investigation

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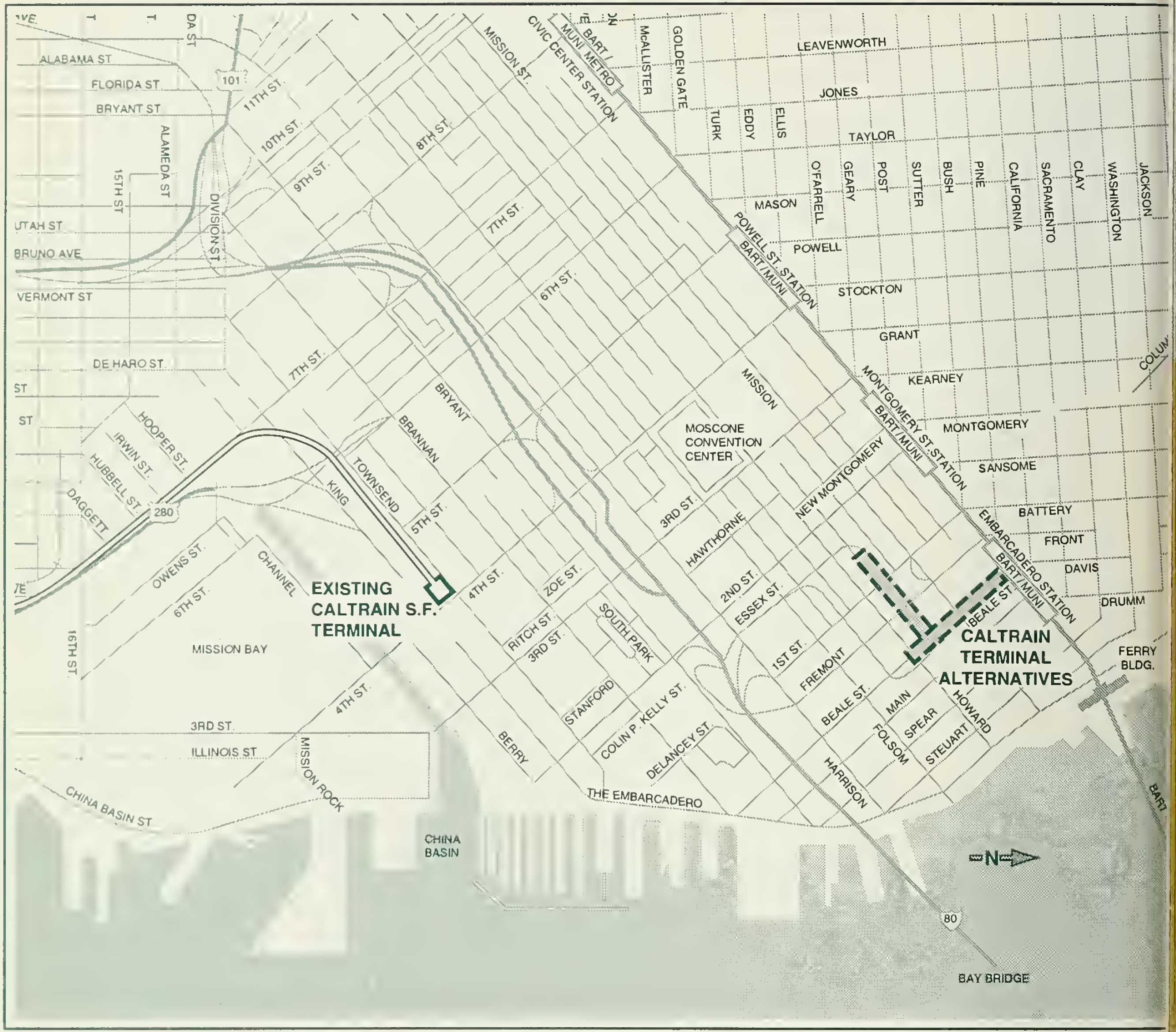
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FINAL REPORT
EOTECHNICAL SITE INVESTIGATION
IN S.F. DOWNTOWN STATION RELOCATION
EIS/EIR PROJECT
VOLUME I
MAIN REPORT

Job No. 30751-001-003
September 25 , 1995

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September 25, 1995
Job No. 30751-001-003

ICF Kaiser Engineers, Inc.
1800 Harrison Street
Oakland, CA 94612-3430

Attention: Mr. David Minister
Senior Project Manager

Gentlemen:

Final Report
Geotechnical Site Investigation
CalTrain S.F. Downtown Station Relocation
San Francisco, California

We are pleased to submit 40 copies of the referenced report presenting the results of the site investigation for the CalTrain project.

The work was undertaken at the request of ICF Kaiser Engineers, Inc. and is substantially in accordance with our proposal dated April 10, 1995 and our Contract No. 65928-SC-03 with ICF Kaiser Engineers, Inc.

A draft of this report was submitted for your review and comments on July 17, 1995. The final report incorporates all the comments received from you.

Very truly yours,

DAMES & MOORE

D. Koutsoftas

D. Koutsoftas
Principal

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FINAL REPORT
GEOTECHNICAL SITE INVESTIGATION
CALTRAIN S.F. DOWNTOWN STATION RELOCATION EIS/EIR PROJECT

1.0 INTRODUCTION

This report presents the results of the Geotechnical Site Investigation program performed between April and June 1995 and between July and August 1995 for the CalTrain S.F. Downtown Station Relocation project. The investigation is part of the EIS/EIR effort for the CalTrain project. The work was performed at the request of ICF Kaiser Engineers, Inc. and is in accordance with the scope of services outlined in our proposals dated April 10, 1995 and July 25, 1995; and ICF Kaiser Engineers, Inc. Contract No. 65928-SC-03, and Amendment No. 1 thereof.

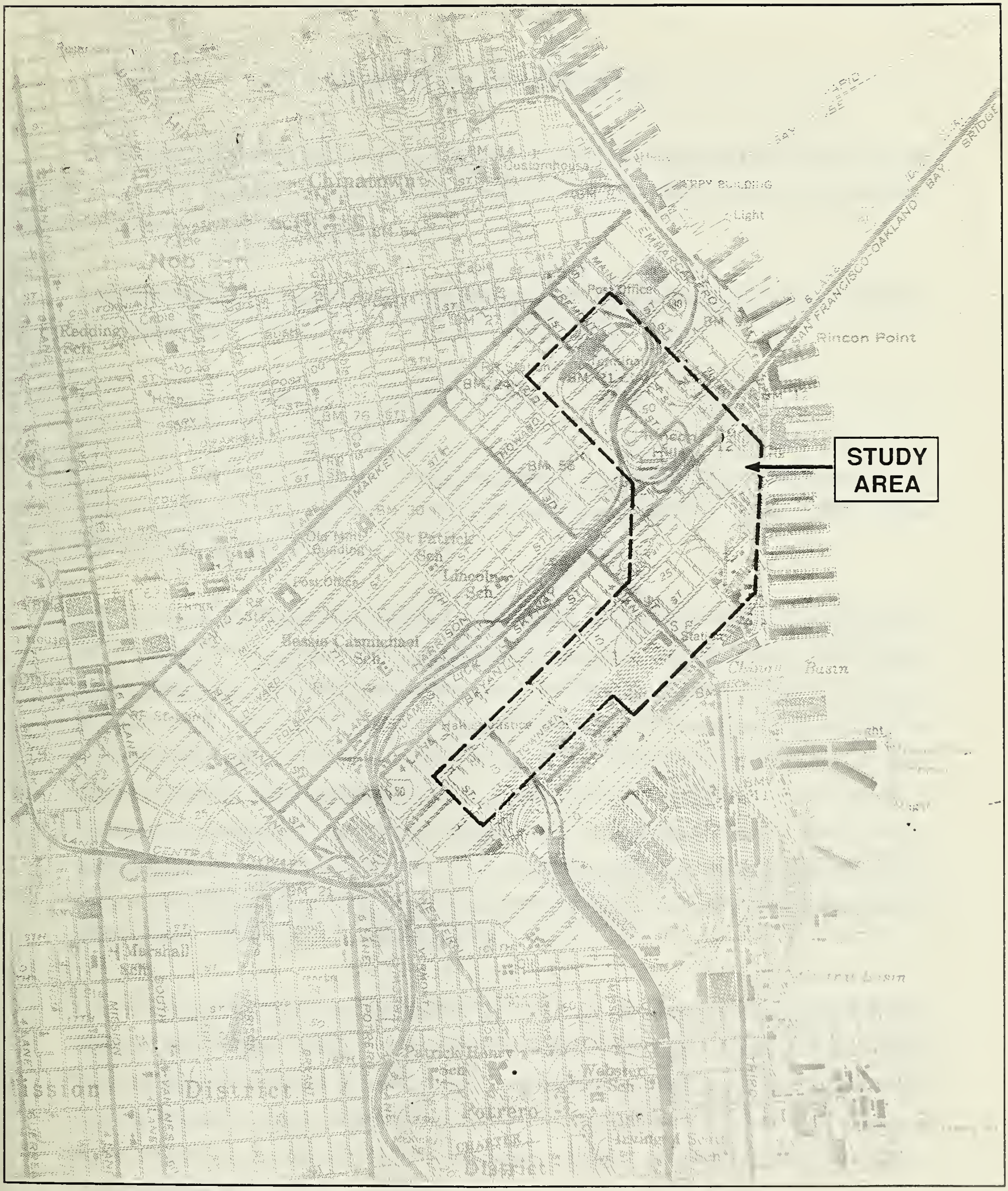
The site of the study corridor is located south of Market Street as shown on Plate 1-1. In the north-south direction the alignment extends from Market Street to King Street, between Beale and Second streets. In the east-west direction the study area covers a strip extending from the waterfront along the Embarcadero to Sixth Street, between King and Brannan streets. Between Sixth and Seventh streets, the three routes merge into a single route that connects to the existing CalTrain tracks along Seventh Street.

The study involves evaluation of various alternative alignments and the environmental, technical, and cost feasibility of various construction alternatives, including cut-and-cover tunneling under existing streets and mined or bored tunnels along at least portions of the various alignments. In addition to the construction of the railroad line required for the proposed relocation, the project involves construction of a downtown station. Three alternatives for the station are being considered at the present time. The first involves the construction of a deep underground station along Beale Street between Mission and Market streets that would connect to the existing BART Embarcadero station. The second alternative consists of constructing a shallower underground station along Beale Street between Folsom and Mission streets with a connecting underground walkway to the Embarcadero BART station. The third option is to construct a station at the existing Transbay Transit Terminal.

This report presents the results of the site investigation work, consisting primarily of the collection of subsurface data from boreholes previously drilled in the vicinity of the project corridor, as well as from boreholes drilled for this investigation. The results of laboratory tests and the subsurface stratigraphic information have been reviewed and evaluated to develop the necessary information for conceptual design purposes. This report presents factual data regarding the

subsurface soils and rocks that will be encountered along the study corridor and presents interpretation of the subsurface stratigraphy and geotechnical characteristics of the major soil strata and rock formations. Design criteria and recommendations as well as discussion of appropriate construction methods and impacts of the proposed construction are presented in a separate "Geotechnical Findings" report.

Following this Introduction (Section 1); a brief description of the proposed construction in Section 2; the definition of the purpose and scope of the investigation in Section 3; and a review of the geologic and site history in Section 4, the report consists of two main parts. Section 5 summarizes the results of the field and laboratory work, and Section 6 provides our evaluation of subsurface conditions with particular reference to the various alignment alternatives.



Source: U.S.G.S. 7.5 minute San Francisco North quadrangle, 1973.

VICINITY MAP

September 1995
30751-001-003

ICF Kaiser Engineers, Inc.
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

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PLATE 1-1

2.0 PROPOSED CONSTRUCTION

The Peninsula Corridor Joint Powers Board (JPB) plans to extend the Peninsula Commute Service (PCS) from its present remote terminal at Fourth and Townsend Streets to the Financial District in downtown San Francisco.

At present the following alternative alignments and station locations are being considered, as shown on Plate 2-1.

1. CalTrain Terminal Station Locations

1.1 The CalTrain Terminal would be located at Market and Beale Streets: an approximately 1,000-foot-long, 3-level cut-and-cover subway under Beale Street (Alternatives 2A and 2A′).

1.2 The CalTrain Downtown San Francisco Terminal would be located all on *one* level below ground in a staggered track and platform arrangement between Market and Folsom streets using both the Beale Street right-of-way and the vacant Caltrans right-of-way east of Beale Street. Two of the CalTrain Terminal tracks would extend underground all the way to Market Street with a mezzanine above. The other two CalTrain Terminal tracks would be located at the same elevation as the first two, but would only extend as far as Mission Street utilizing the Caltrans right-of-way east of Beale Street. An attractive underground mezzanine would connect the two sets of tracks and a moving sidewalk would connect the Mission Street tracks to the BART/MUNI Embarcadero Station at Market and Beale streets. This is part of Alternative 2B.

1.3 The CalTrain Terminal would be located all on one level below ground between Mission and Folsom streets, using both the Beale Street right-of-way and the vacant Caltrans right-of-way east of Beale Street. An attractive underground mezzanine with a moving sidewalk would connect the north end of the CalTrain terminal at Mission and Beale streets to the BART/MUNI Embarcadero Station at Market and Beale streets (Alternative 2C).

1.4 The CalTrain Terminal would be located on the second level of a new Transbay Transit Terminal (TTT) to be constructed at the site of the existing terminal, after the existing TTT has been demolished (Alternative 3A).

1.5 The CalTrain Terminal would be located one level below ground in a new Transbay Transit Terminal, which would be constructed at the site of the existing terminal after the existing TTT has been demolished (Alternative 3B).

2. CalTrain North-South Alignments

2.1 Along Beale Street from Market Street to the Embarcadero to be constructed by cut-and-cover tunneling techniques. This is part of Alternative 2A-C.

2.2 Along Beale Street from Market to Harrison streets constructed by cut-and-cover tunneling techniques. Southward from Harrison Street, this alignment would be constructed as a mined or bored tunnel in a sweeping, approximately 3,000-foot-radius curve under private property to either Third and Townsend streets or Third and King streets. This is part of Alternative 2A′-M.

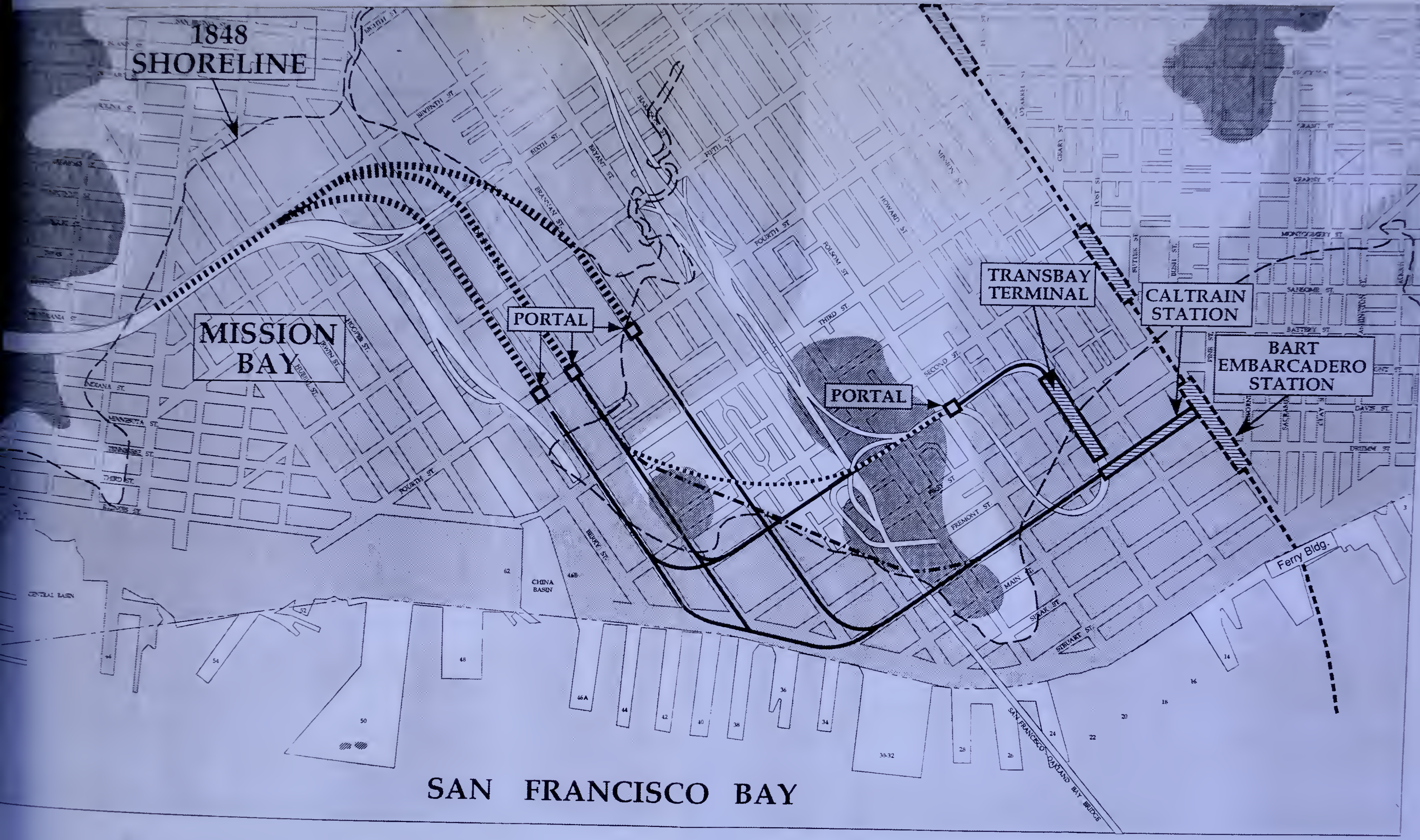
2.3 Colin P. Kelly Jr./Essex Street Alignment from Howard Street to either King or Townsend Street. The portion of this alignment between Folsom and Townsend streets would be constructed as a bored or mined tunnel under private property and the remainder as a cut-and-cover. The length of the tunnel portion remains to be determined. The portal would be near Folsom Street. The tunnel portion would connect to an aerial structure that would extend north to the existing Transbay Transit Terminal. This is part of Alternative 3B. Another variation of this alternative involves an underground line north of Folsom Street connecting to an underground station at the Transbay Transit Terminal, one level below ground (Alternative 3B). Two options are being considered for the portion of the alignment that will be constructed as a mined tunnel. The first alternative would involve tunneling between Folsom Street to the north and a point midway between Brannan and Bryant streets at the south end. The second option would involve a longer tunnel that would extend from Folsom Street to Bryant Street under Essex and Colin P. Kelly Jr. streets, and then follow a sweeping curve under private property between the intersection of Colin B. Kelly Jr. and Bryant streets and the intersection of Third and Townsend Streets (stations 172+30 and 195+00).

3. CalTrain East-West Alignments

3.1 Along Townsend Street (the Preferred Alignment) from Seventh Street to the Embarcadero; and then along the Embarcadero to the north-south alignment along Beale Street. This portion would be constructed by cut-and-cover tunneling methods with the

portal between Fifth and Fourth streets. An extended subway with a portal near Seventh and Berry streets will also be considered (longer subway through Mission Bay).

3.2 Along King Street from Seventh Street to the Embarcadero, and then along the Embarcadero to the north-south alignment along Beale Street. This portion would be constructed by cut-and-cover tunneling methods, with a portal between Fourth and Fifth streets. An extended subway with a portal near Seventh and Berry streets will also be considered (longest subway segment through Mission Bay).



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Scale in Feet

KEY

▨	Fill
▩	Rock Outcrop

SITE PLAN AND PROPOSED ALTERNATIVES

ICF Kaiser/De Leuw
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

September 1995
30751-001-003

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PLATE 2-1

3.0 PURPOSE AND SCOPE

The purpose of the geotechnical investigation is to develop an appropriate understanding of the ground conditions that may be encountered along each of the alignments. This information will be the basis for evaluation of potential construction methods, development of preliminary geotechnical criteria for conceptual design, and a general assessment of impacts on adjacent structures, streets, utilities and other facilities that may be encountered along the alignment. The work associated with these assessments will be presented in a separate report.

To achieve the objectives of the geotechnical engineering task we undertook the following scope of services:

Task No. 1 Collection, Review, and Analysis of Existing Geotechnical Data

The first step in this task was to collect available geotechnical data from previous projects along the proposed alignments. The information was compiled, reviewed, and analyzed with the objective of subdividing each alignment into various sections of similar subsurface conditions, and/or where similar construction methods may be used.

Dames & Moore and AGS, Inc., who assisted Dames & Moore in data collection, used data available in their files from a number of projects performed in the vicinity of the proposed alignments. The existing data were supplemented by whatever information was available in public documents at City Hall (as part of construction permit applications), from other sources such as Caltrans projects, and from other geotechnical firms that made available to us existing subsurface data from their files.

The information was reviewed and summarized with regard to depths and thicknesses of the major soil strata, depth to groundwater level, depth to the top of rock, and rock types and quality within the depth of interest.

The information generated from this subtask was combined with the results of the exploration program (Task No. 2, see below) to characterize the subsurface conditions along each of the alignments.

Task No. 2 Subsurface Exploration Program

The subsurface exploration program was planned with the objective of providing subsurface data at strategically selected locations to supplement the existing data collected in Task 1, and of providing samples for laboratory testing of the major soil strata and rocks that will be encountered along the proposed tunnels and excavations.

The exploration program consisted of:

- Drilling ten boreholes and obtaining samples of soils and rock cores for examination and laboratory testing. An additional borehole (B-6) was initially planned to be drilled along the Embarcadero near Brannan Street but was abandoned because it was not possible to obtain the necessary permits from the Port of San Francisco, the owner of the property.
- Performing vane shear tests at five locations to evaluate the strength of the soft Bay Mud deposits, which is critical to the design of the shoring system for the excavations. The initial program included one additional vane shear test (FV-1) that had to be abandoned because we could not obtain the necessary permit from the Port of San Francisco.

Plate 3-1 shows the locations of the boreholes and field vane tests. The depths of the boreholes ranged from 52 to 150 feet below ground surface.

A health and safety plan was developed to provide guidelines for preliminary monitoring and detection of potentially contaminated soils which might be encountered during the drilling, sampling, or coring operations, so that appropriate health and safety precautions could be implemented, if necessary.

All necessary permits for drilling on city streets and/or sidewalks, as well as traffic and health permits, were obtained by AGS, Inc., who were responsible under subcontract to ICF Kaiser Engineers, Inc. to supervise the drilling and sampling operations.

The boreholes were drilled by Pitcher Drilling Company under subcontract to Dames & Moore. Pitcher Drilling Company is a certified Disadvantaged Business Enterprise (DBE). The

field vane shear tests were performed under the direct supervision of Dames & Moore, with Pitcher providing the drilling equipment and support to perform the tests.

The disposal of the drill cuttings was coordinated by Pitcher Drilling Company, who issued a subcontract to DECON Environmental Services, Inc. AGS, Inc., who provided field supervision of the drilling and sampling activities, oversaw the collection and storage of the drill cuttings prior to disposal. DECON Environmental Services, Inc. is responsible for testing and disposal of the drill cuttings in accordance with applicable local, state and federal regulations.

During drilling, the cuttings were collected and stored in drums provided by Pitcher Drilling Company. At the end of each day, Pitcher Drilling Company staff transported the cuttings and placed them in a special storage bin provided by DECON Environmental Services, Inc. At the end of the drilling work, DECON Environmental Services, Inc. collected appropriate samples from the bin for chemical testing. Once the results of the required chemical analyses become available, DECON Environmental Services, Inc. will determine how to dispose of the cuttings.

Task No. 3 Laboratory Testing

Soil samples and rock cores recovered from the exploratory boreholes were transported to our San Francisco laboratory for further examination and testing. Representative samples from the major soil strata and rock types encountered were selected for laboratory testing to evaluate their physical and engineering properties.

The laboratory program included the following types and numbers of tests:

Soil Testing

1. 40 Moisture and density tests
2. 2 Consolidation tests on samples of cohesive soils
3. 5 Unconsolidated undrained triaxial compression tests to evaluate the shear strength of cohesive soils
4. 23 Grain size tests on cohesionless soils

5. 4 Fines content tests (percentage passing the No. 200 sieve)

6. 10 Atterberg Limit tests on cohesive soils for identification purposes

7. 9 Consolidated drained direct shear tests to evaluate the strength characteristics of cohesionless soils

Rock Testing

1. 29 Density tests

2. 27 Point load tests, which serve as a strength index test and are useful in assessing excavatability of rock

3. 16 Unconfined compression tests to determine the strength of rock samples

4. 8 Unconsolidated undrained triaxial compression tests to evaluate the strength of soft rock

Task No. 4 Review, Analysis, and Evaluation of the Data

The information collected in Task No. 1 and the results of the field explorations and testing were reviewed, evaluated and synthesized to develop stratigraphic sections along the various alignments.

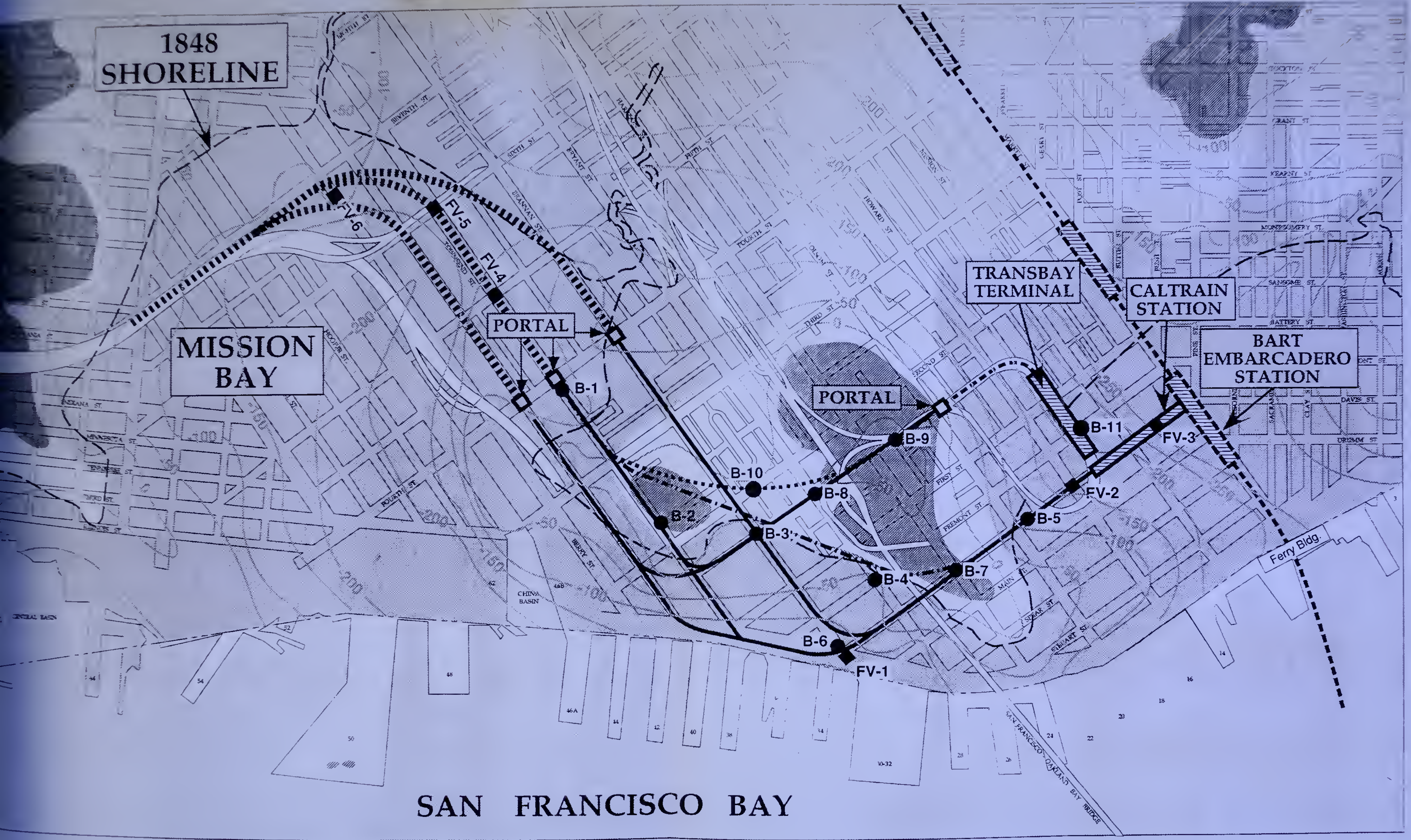
The results of the field and laboratory tests were summarized and evaluated, so that characteristic properties could be estimated for each of the major strata to facilitate the engineering analyses.

Task No. 5 Preparation of Report

The results of the site investigation were summarized and this report was prepared. The report includes the following information:

- Logs of boreholes and photographs of rock cores, presented in Appendix A.

- The results of laboratory tests, presented in Appendix B.
- Stratigraphic sections along the various alignments (Plates 6-2 through 6-11).
- Analysis and evaluation of the field and laboratory data, review of the site geology and seismicity, and review of the geotechnical characteristics of the major strata (Sections 4,5 and 6).



KEY

● **B-5** Boring Location

■ **FV-3** Field Vane Location

┈ Bedrock Contours (feet)

▭ Fill

▨ Rock Outcrop

BORING LOCATION PLAN

September 1995
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PLATE 3-1

4.0 GEOLOGIC SETTING

4.1 REGIONAL GEOLOGIC SETTING

The site is located in the northeast portion of the San Francisco peninsula, which is bounded to the west by the Pacific Ocean, to the east by San Francisco Bay, and to the north by the Golden Gate. The peninsula is located in the Coast Ranges geomorphic province, although the dominant northwesterly trend of ridges and valleys characteristic of the Coast Ranges is obscured in San Francisco except for minor features such as Russian Hill and Telegraph Hill. San Francisco Bay lies within a down-dropped structural block bounded by the East Bay Hills and the Santa Cruz Mountains (Sedlock, 1995). The northern portion of the San Francisco peninsula also lies within this structural block. A regional geologic map of the San Francisco Bay area is shown on Plate 4-1. As shown on the map, other major structural elements of the San Francisco Bay region include the San Andreas fault, located approximately 8½ miles west of the site, and the Hayward fault, which is approximately 9 miles east of the site.

The regional topography of the San Francisco peninsula is characterized by relatively rugged hills formed by Jurassic- to Cretaceous-aged bedrock surrounded by low flat-lying areas which are underlain by Quaternary sedimentary deposits. Bedrock consists of highly deformed and fractured sedimentary rocks of the Franciscan assemblage. The present topography resulted from the Coast Range orogeny, which occurred during the late Pliocene and Pleistocene epochs (Norris and Webb, 1990). Locally, Franciscan bedrock contains large through-going shear zones such as the City College and Fort Point–Potrero Hill–Hunters Point shear zones (Plate 4-2).

In the peninsula area, Quaternary sediments lie unconformably on the eroded bedrock surface, and record a sequence of sea-level rise and fall throughout the mid- to late-Pleistocene. The lowermost sediments were deposited in a terrestrial environment, probably associated with low sea-level stands during the Illinoian and early glaciations. These sediments are overlain by estuarine deposits referred to as the Yerba Buena Mud (also known locally as Old Bay Clay deposit), which formed during the high sea-level stand of the Sangamon interglacial (Sloan, 1992). The Yerba Buena Mud was deposited in an estuarine environment similar to that which presently exists in San Francisco Bay, and therefore has a lateral distribution similar to that of the modern bay estuaries.

Sea level lowering associated with the onset of Wisconsin glaciation exposed the Yerba Buena Mud and resulted in erosion of the estuarine sediments and subsequent deposition of alluvial and eolian sediments. Deposition of the terrestrial sediments continued until the bay was once again inundated by rising sea level associated with the end of the Wisconsin glaciation. This most recent

sea level rise resulted in the sea entering the Golden Gate 10,000 to 11,000 years ago (Atwater et al., 1977). Sea level rise was most rapid during the early stages of the transgression, but decreased in rate to 0.1 to 0.2 centimeters per year over the last 6,000 years (Atwater et al., 1977). The return of the bay resulted in deposition of estuarine sediments, which continue to accumulate in the present bay. These estuarine sediments are referred to as Bay Mud. Historical development of the San Francisco Bay area has resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands and creek beds in an effort to reclaim land.

4.2 SITE GEOLOGIC SETTING

Stratigraphic units beneath the study area include artificial fill material, unconsolidated Quaternary sediments, and Franciscan assemblage bedrock. Quaternary sediments overlie bedrock throughout the study area except for Rincon Hill, which is the only surface exposure of Franciscan bedrock in the study area. Bedrock is less than 10 feet below ground surface near Townsend and Second streets.

Deposits of artificial fill overlie the younger Bay Mud and typically form the surface exposures in the study area. Fill material consists of clay, sand, and rock fragments and contains various debris such as concrete, bricks, and wood. The clays contain silt, sand, and/or gravel and are typically soft to medium stiff. The sands contain varying amounts of silt and gravel. The thickness of fill is generally less than 30 feet except in areas adjacent to the sea walls along the Embarcadero where fill thicknesses are up to 45 feet. Much of the sandy fill originated from sand dunes that used to cover much of San Francisco.

As discussed in Section 4.1, Quaternary sediments beneath the study area consist of an alternating sequence of terrestrial and estuarine deposits which reflect major sea level fluctuations during the Pleistocene and Holocene epochs. The uppermost Quaternary deposits in the study area include undifferentiated sediments between Rincon Hill bedrock exposures and the 1848 shoreline, and Bay Mud which underlies fill material in areas that are generally bayward of the 1848 shoreline. Bay Mud extends landward of the 1848 shoreline near the Transbay Terminal and in the southern portion of the study area between Third and Seventh streets in areas of former marshlands. The approximate extent of Bay Mud deposition corresponds with the distribution of fill shown on Plate 4-3.

The undifferentiated sediments probably consist of discontinuous units of clayey to sandy Colluvium, Alluvium, and Dune Sands that are expected to have a maximum combined thickness of 10 to 20 feet in the study area. Bay Mud typically consists of olive gray to dark gray fat clay with

small but variable amounts of organics and sand. Bay Mud was deposited in an estuarine environment within the last several thousand years of the current high stand of sea level (Holocene epoch). Within the study area, the thickness of Bay Mud ranges from a few feet along the 1848 shoreline to a maximum of approximately 25 feet along the Embarcadero near Townsend Street, and up to 40 feet near the Transbay Terminal, and along Beale between Mission and Howard streets.

Bay Mud is underlain by brown to reddish-brown sand and clayey or silty sands which are interpreted by Schlocker (1974) to belong to the Colma formation. The Colma sediments probably include alluvial, estuarine, and eolian deposits likely formed during the last low stand of sea level (Wisconsin glacial) (Schlocker, 1974). These deposits interfinger with the undifferentiated Quaternary deposits discussed above, and have a total thickness which ranges from 20 to 50 feet in the study area.

Beneath the Colma formation is a relatively thick deposit referred to as "Old Bay Clay" and, more recently, the Yerba Buena Mud (Sloan, 1992). This deposit consists of moderately stiff, overconsolidated low to medium plasticity clay (Schlocker, 1974). Old Bay Clay was deposited during the Sangamon interglacial (roughly 100,000 years ago) within the same geologic environment as Bay Mud (Atwater et al., 1977; Sloan, 1992). It ranges in thickness beneath the study area from a few feet near the original (Sangamon) shoreline, which was probably roughly parallel to the inland limit of Bay Mud deposition, to a maximum of approximately 20 feet along the Embarcadero near Townsend Street, and 130 feet near the Transbay Terminal. As with Bay Mud, Old Bay Clay extends landward of the 1848 shoreline near the Transbay Terminal and in the southern portion of the study area between Third and Seventh streets.

Pleistocene sand and clay deposits, probably associated with a low sea-level stand during the Illinoian glaciation, were observed in several borings beneath the Old Bay Clay. These deposits are probably similar in nature to the younger Colma formation sediments overlying Old Bay Clay. Within the study area, these sand and clay deposits reach a maximum thickness of roughly 100 feet along the Embarcadero, but are generally less than 70 feet thick elsewhere.

The Franciscan assemblage in the study area is composed of massive to thinly bedded Sandstone, thinly interbedded Sandstone and Shale, and Siltstone. These rocks are bounded to the southwest by the Fort Point–Potrero Hill–Hunters Point shear zone (Plate 4-2), and are locally deformed and highly sheared within the study area. Bedrock bedding appears to be tilted to the east and southeast at an approximate average dip of 50 degrees. Bedding attitudes for bedrock at Rincon Hill are shown on the surficial geologic map of the study area (Plate 4-3). Fracture orientations and spacings are highly variable, with fracture spacing generally less than a few feet. The rock strength

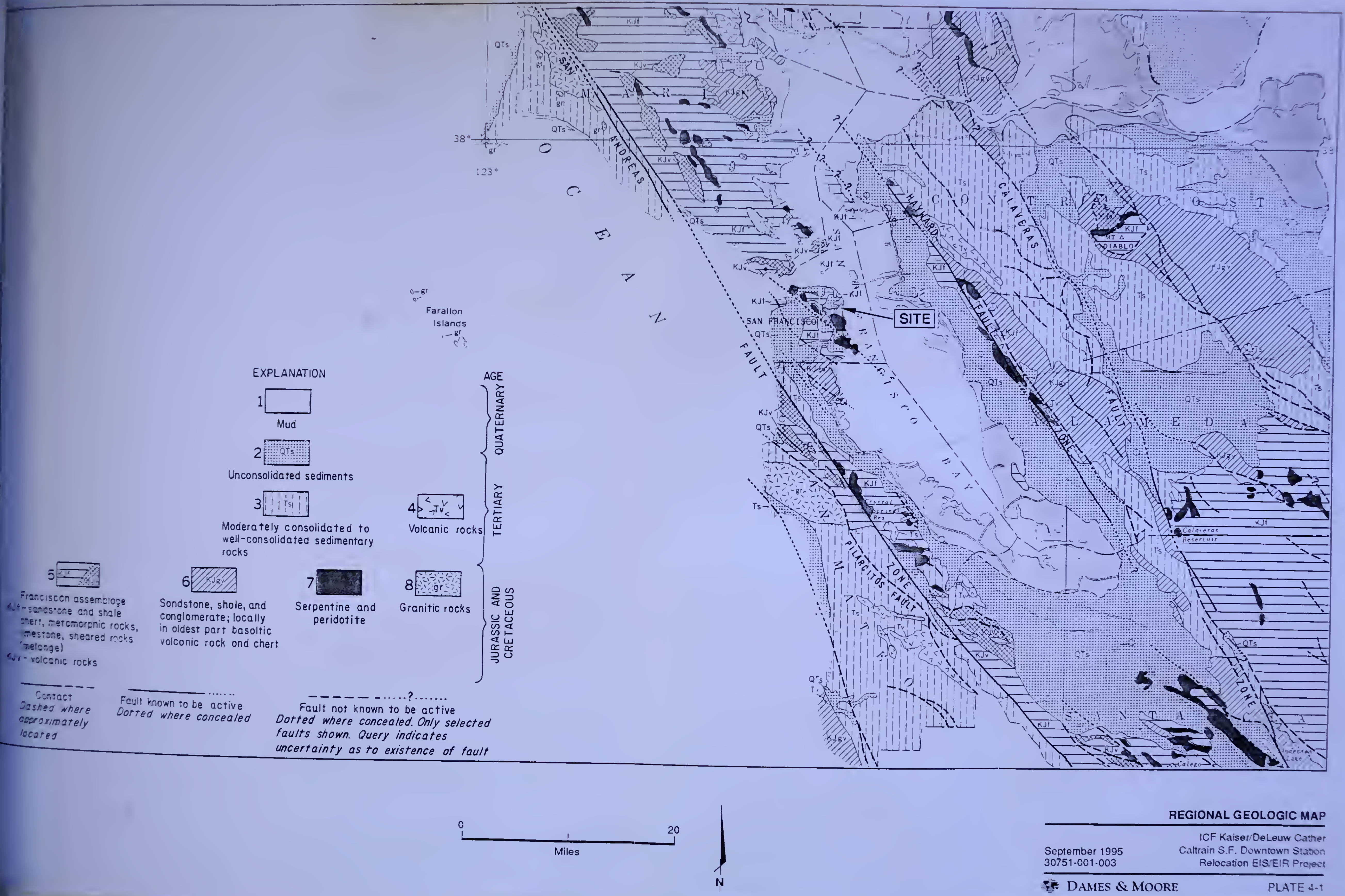
varies from high for the massive sandstone to low or very low for shales and highly sheared shale and sandstone. The top of bedrock represents an erosional surface that pre-dates deposition of the lowermost Quaternary sediments.

First groundwater is between 7 and 10 feet below ground surface in the flat-lying portions of the study area, which corresponds to elevations of -3 to -10 feet relative to City and County of San Francisco Datum (8.6 feet mean sea level). Beneath Rincon Hill, the water table probably perched is expected to be higher. Bedrock under Rincon Hill has a low permeability except where highly fractured (Schlocker, 1974). Permeabilities of the fill materials and Quaternary sediments range from high for well-sorted sands within the Colma formation to very low for Bay Mud and Old Bay Clay.

4.3 SEISMICITY

The study area is located within a region characterized by the seismically active San Andreas fault system, which is the principal tectonic element of the North American/Pacific plate boundary in California. In the San Francisco Bay area, seismic slip is partitioned onto subsidiary structures, such as the Hayward and Calaveras faults, that are distributed across the Coast Ranges province. Plate 4-4 shows the major active faults in the San Francisco Bay area. Many of these faults have been active in historical time, while earthquakes on other major faults have not been recorded.

The San Andreas and Hayward faults have the highest slip rates and are the most seismically active of any faults in the Bay Area. Other important earthquake sources that are capable of producing large magnitude earthquakes are the San Gregorio, Calaveras, Rodgers Creek, and Greenville fault zones. Historically, the most active components of the fault system are: the San Andreas fault, source of the 1906 Magnitude 8.2 and 1989 Magnitude 7.1 earthquakes; the Hayward fault, source of the 1836 and 1868 Magnitude 6.8 earthquakes; and the Calaveras fault, source of the 1911 Magnitude 6 and 1984 Magnitude 6.2 earthquakes.


SAN ANDREAS FAULT
HAYWARD FAULT ZONE
CALAVERAS FAULT ZONE
PILARCITOS FAULT ZONE
SAN FRANCISCO BAY
OCEAN
MARIN
CONTRA COSTA
ALAMEDA
SAN MATEO
SANTA CLARA
SAN FRANCISCO
SITE
MT DIABLO
Farallon Islands
Crystal Springs Res
Calaveras Reservoir
38°
123°
QTs
Tsl
Ts
Tv
KJf
KJv
KJgv
gr
EXPLANATION
AGE
1 Mud
2 QTs Unconsolidated sediments
QUATERNARY
3 Tsl Moderately consolidated to well-consolidated sedimentary rocks
4 Tv Volcanic rocks
TERTIARY
5 Franciscan assemblage KJf - sandstone and shale chert, metamorphic rocks, limestone, sheared rocks (melange) KJv - volcanic rocks
6 KJgv Sandstone, shale, and conglomerate; locally in oldest part basaltic volcanic rock and chert
7 Serpentine and peridotite
8 gr Granitic rocks
JURASSIC AND CRETACEOUS
Contact Dashed where approximately located
Fault known to be active Dotted where concealed
Fault not known to be active Dotted where concealed. Only selected faults shown. Query indicates uncertainty as to existence of fault
0
20
Miles
N

REGIONAL GEOLOGIC MAP

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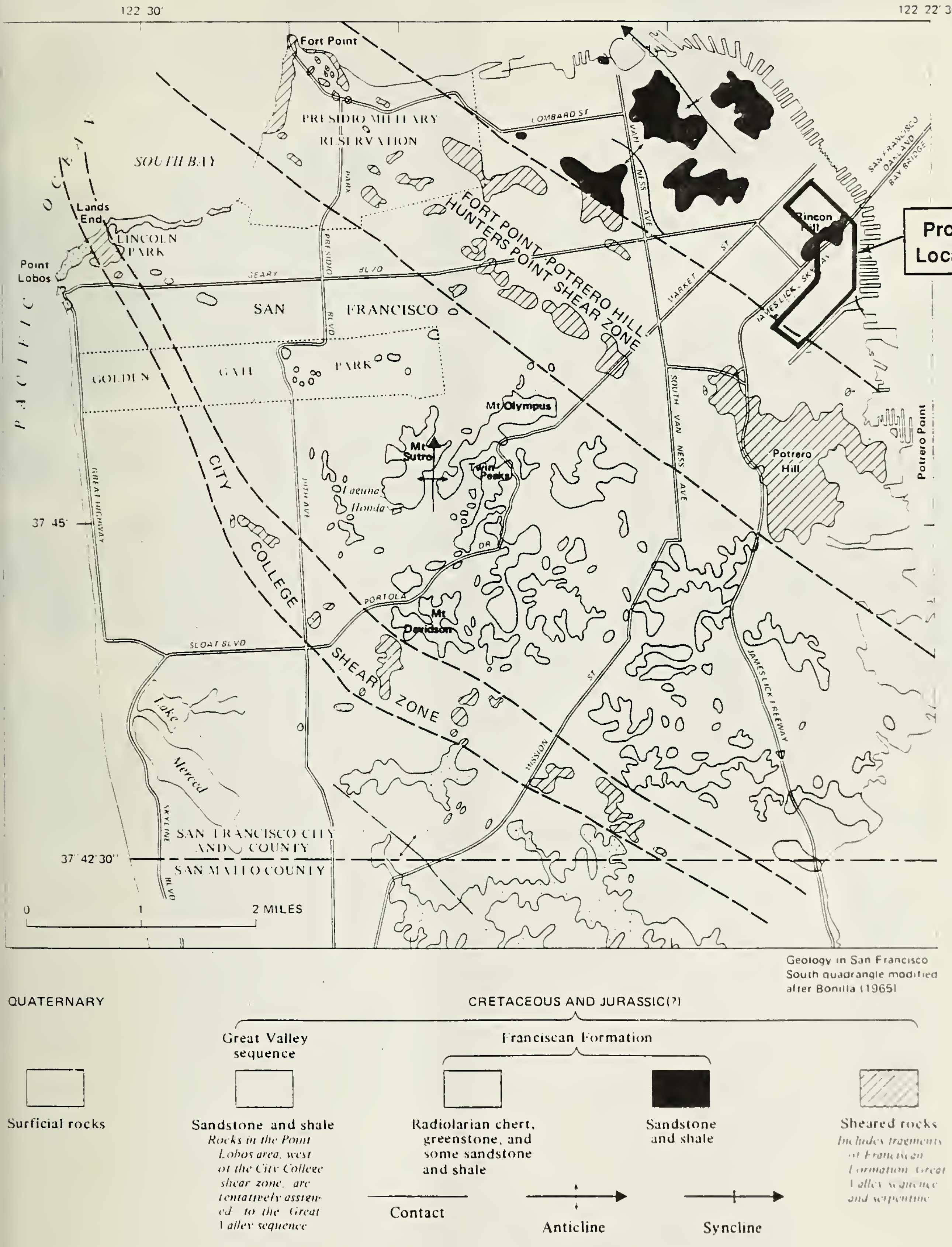
PLATE 4-1



Geology in San Francisco South quadrangle modified after Bonilla (1965)

Source: Schlocker, 1974

GENERALIZED GEOLOGY OF SAN FRANCISCO

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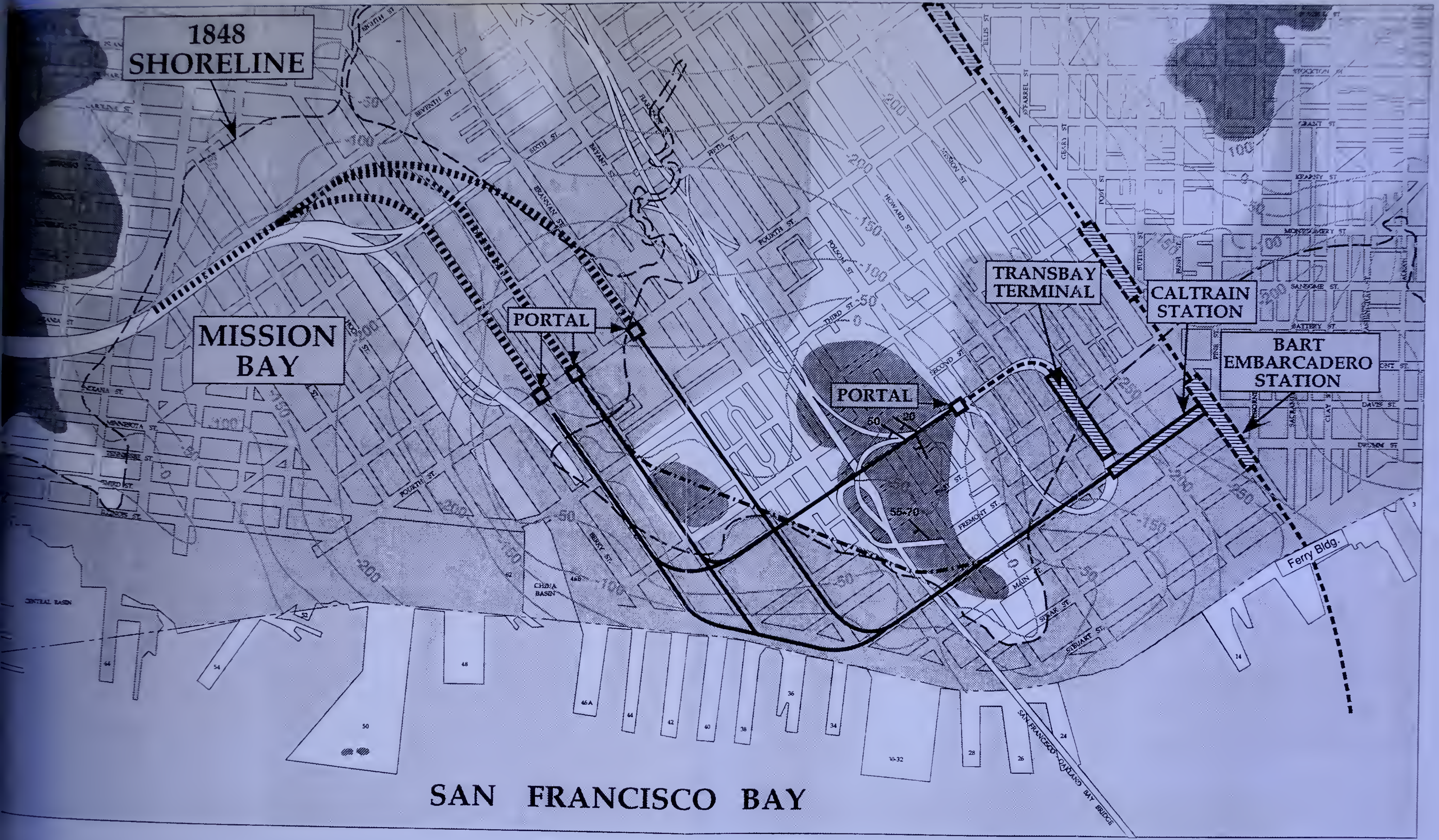
PLATE 4-2



KEY

Bedrock Contours (feet)

Fill

Rock Outcrop

50 Strike and Dip of Bedrock Bedding

GENERALIZED GEOLOGY OF THE STUDY AREA

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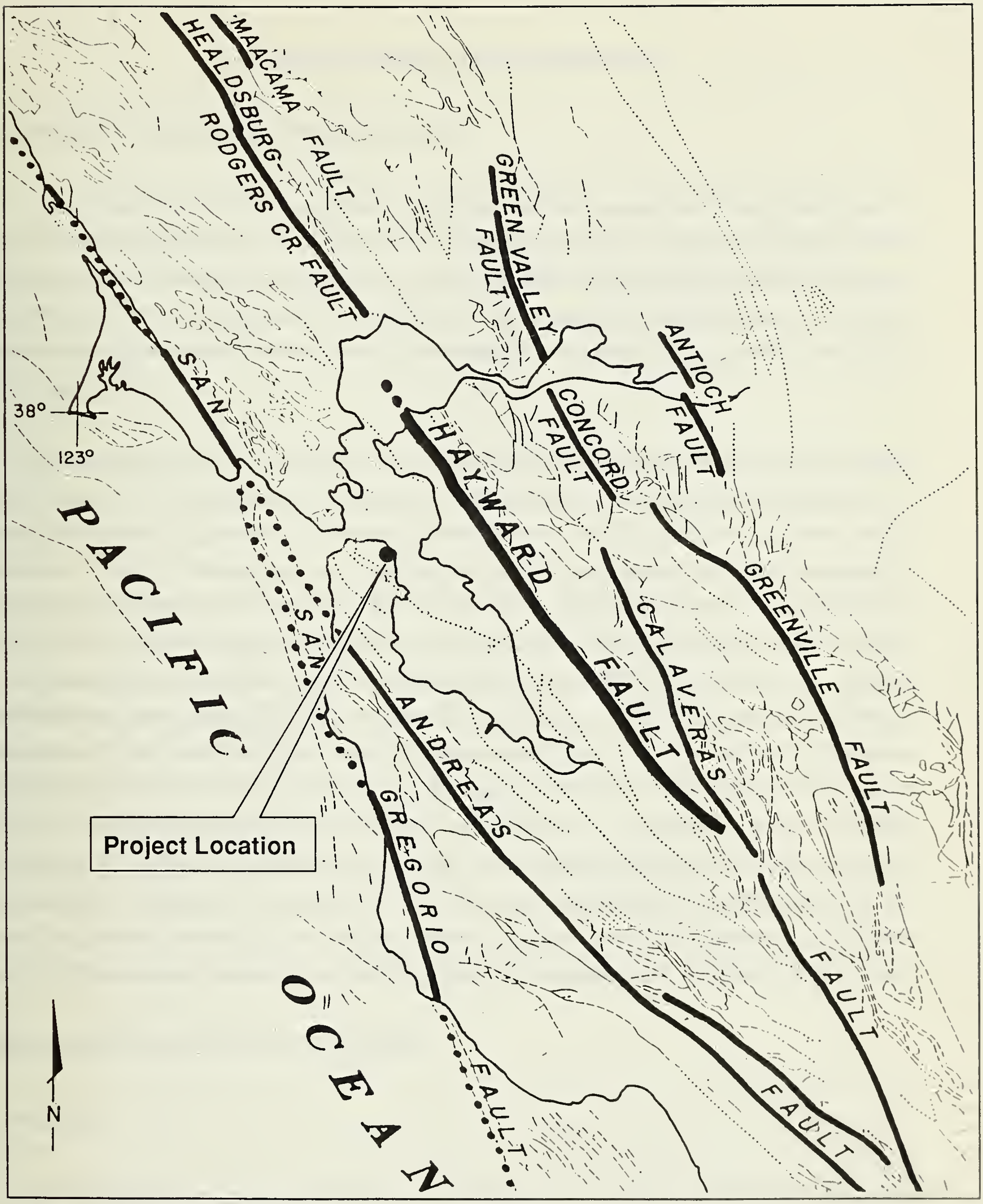
PLATE 4-3



Source: California Division of Mines and Geology, Special Publication 78 (1987)

LOCATIONS OF ACTIVE FAULTS IN THE SAN FRANCISCO BAY AREA

September 1995
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CalTrain S.F. Downtown Station
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DAMES & MOORE

PLATE 4-4

5.0 GEOTECHNICAL EXPLORATION

5.1 REVIEW OF EXISTING INFORMATION

Concurrently with the field exploratory program, available geotechnical data from previous projects along the alignments were collected. The existing information was reviewed and analyzed with the objective of complementing the information collected from the field exploration program. The existing data were then integrated with the information obtained from the exploratory program to subdivide the alignments into sections with similar subsurface conditions where similar construction methods may be used.

Subsurface data were collected from our files and from the files of AGS, Inc, who assisted Dames & Moore in data gathering. Data were collected from various projects performed in the vicinity of the proposed alignments. Collected information was carefully reviewed and screened for completeness. Over 170 boring logs were selected and used to complement our field investigation. Plate 5-1 shows the locations of the borings that were reviewed. Borings are labeled sequentially and divided into four general categories according to their location. Borings located between Market and Harrison Streets are labeled with the letter F (for the Foot of Market Street area). Borings located within the area bounded by Harrison Street, Third Street, and the Embarcadero are labeled with the letters SB (for the South Beach area). Borings located between Third and Sixth Streets are labeled with the letters SM (South of Market). Borings located west of Sixth Street and South of Townsend Street are labeled with the letters MB (Mission Bay). A sequential number was added after the designation letters to identify each boring. Information obtained from the evaluation of existing borings, in addition to the subsurface information gathered from our exploratory program, was used to generate subsurface stratigraphic sections along the proposed alignments. A detailed discussion of the subsurface stratigraphy along the alignments is presented in Section 6 of this report.

5.2 FIELD EXPLORATION AND TESTING

5.2.1 Program

The field exploration program consisted of soil borings, rock coring, and field vane tests. The exploratory program included:

- Drilling ten borings with frequent soil sampling and rock coring for examination and laboratory testing.

- Performing field vane shear tests at five locations to evaluate the strength of the soft Bay Mud deposits.

The locations of the completed exploratory borings and field vanes are shown on Plate 5-2, Exploration Location Plan. The depths of the boreholes ranged from 52 to 150 feet below ground surface. The exploration locations were selected to provide information to supplement the available existing data and to provide coverage in the vicinity of the proposed alignments. While fulfilling these criteria, the locations of the exploratory borings were chosen to avoid underground utilities, surface obstructions, to clear overhead structures, and to comply with traffic closure restrictions. Due to these constraints, the actual boring locations and field tests shown on Plate 5-1 are somewhat different from those indicated in our proposal dated April 10, 1995 and in our additional scope of work dated July 3, 1995. Detailed descriptions of the drilling equipment, procedures, and the results for each of the field tests are presented in Appendix A. The results of the site investigation program are presented below.

5.2.2 Borings

Ten borings were drilled as part of this contract to aid in the evaluation of the subsurface stratigraphy along the proposed alignments and to obtain samples for laboratory testing. Plate 5-2 shows the locations of the borings and field vane tests in relation to the proposed alignment alternatives.

Pitcher Drilling Company, of East Palo Alto, California, drilled the borings under the supervision of an engineer from AGS, Inc., who maintained records of all field activities, classified the soils and rock encountered, and maintained a continuous log of the borings. Soil samples were generally obtained at 5-foot depth intervals in the fill and underlying soils with various sampler types judged most appropriate for the soil types encountered.

In the current investigation, some of the sampling in the Bay Mud, Old Bay Clay, and Upper Sands with Clay Lenses was performed using a Pitcher sampler, which was generally advanced hydraulically. Bedrock was continuously cored or sampled. Coring was performed using a NQ type barrel (a 2-inch-diameter split diamond bit barrel). Typical cores were 5 feet in length; however, occasionally shorter runs were performed due to poor sample recovered or jamming of rock within the core barrel. Rock samples were also taken with the Pitcher sampler where the rock was found too soft to be cored effectively.

Driving resistances were recorded in the boring where driven samples were taken. Two types of driven samplers were used: the Standard Penetration Test sampler and the Dames & Moore underwater U sampler. The dimensions of the U sampler are approximately 2½ inches inside diameter and 3¼ inches outside diameter A standard hammer weight and drop were used to drive the SPT sampler in accordance with ASTM D 1586-90. The same hammer was used to drive the Dames & Moore U sampler. Dames & Moore sampler blowcounts are typically higher than SPT blowcounts. Nonstandard blowcounts can be converted to equivalent standard blowcounts using a conversion factor. Lacroix and Horn (1972) proposed an equation that considers sampler geometry and hammer weight and drop for converting nonstandard penetration resistances to equivalent SPT values. For the U sampler used in this investigation the corresponding approximate conversion factor is 0.38.

Samples of cohesionless soils were obtained using primarily the standard split spoon sampler in accordance with ASTM Standard D1586-90. Unfortunately, the standard split spoon recovers samples that are highly disturbed, too small, and unsuitable for most types of engineering tests. The Dames & Moore U sampler was used intermittently to recover samples of sufficient size and quality for laboratory tests such as direct shear, triaxial compression, density, and sieve analysis.

Profiles of blowcounts versus depth are presented on Plates 5-3 and 5-4 for the Fill and the sandy deposits that normally underlie Fill and Bay Mud, respectively. The data presented for the Fill include information from this investigation and from MUNI Metro Turnback Project. Penetration resistance in the Fill ranges from about 3 to about 40 blows/foot with no apparent trend in the data indicating a highly variable and heterogeneous character of the Fill material. The data for sandy deposits below Fill and Bay Mud indicate blowcounts in the range of about 20 to more than 90 blows/foot. A distinction was made between driving resistance on clay layers and sands. As shown on Plate 5-4, blowcount data in clay layers are consistently lower than those in sand, as should be expected.

5.2.3 Field Vane Shear Tests

The field vane (FV) tests were performed by Dames & Moore at locations shown on Plate 5-2, designated FV-2 through FV-6. Drilling for the tests was performed by Pitcher Drilling. At the location of the field vane test FV-2, the Bay Mud was much thinner than originally anticipated, therefore, the FV-2 location was converted to a boring after the completion of the field vane test. After reaching the bottom of Bay Mud, field vane FV-6 was converted to a boring and extended to bedrock, which was found at a depth of about 60 feet. The holes were advanced and sampled to depths ranging from 31.0 to 85.5 feet below existing ground surface. Tests were performed using

the Geonor vane shear apparatus. In addition to providing a direct measure of the in-situ undrained shear strength at the tested locations, the vane shear test is an indicator of variations in in-situ shear strength. Undisturbed vane shear tests were performed typically at about 3.3 to 4.0-foot intervals in the Recent Bay Mud deposit. Remolded shear tests were also performed and provide an index of the sensitivity of the Mud. Testing equipment, procedures, and results are described in Appendix A.

Results of the FV tests are summarized on Table 5-1. Undrained shear strengths are plotted versus depth in Plates 5-5A and 5-5B. The thickness of Bay Mud at the locations of the field vanes varies. A more detailed discussion of the results of the field vane tests is presented in Section 6 of this report. The ratio of the undisturbed strength to the remolded strength is, by definition, the sensitivity of the soil. Sensitivities determined from 24 tests ranged from 3.0 to 23.9 with an average value of 8.5. These results indicate that the Bay Mud is moderately to very sensitive. Tests results indicate a consistent trend of increasing strength with depth with variations in the in-situ vane strength due possibly to the presence of sand in the Bay Mud.

5.3 LABORATORY TESTS

5.3.1 Test Program

The laboratory test program included conventional tests to confirm existing information on the physical and engineering characteristics of the Fill, Bay Mud, Upper Sands with Clay Lenses, Old Bay Clay, Lower Sands and Clays, and Bedrock. The test program included the following:

- Index tests, such as moisture content, density, sieve analysis, and Atterberg limits.
- Consolidated drained direct shear tests on cohesionless materials. The purpose of the tests was to provide effective-stress-strength parameters and deformation characteristics of the major cohesionless strata such as Fill and Upper Sands with Clay Lenses.
- Unconsolidated undrained triaxial tests on cohesive samples, such as Bay Mud, Old Bay Clay and Stiff Clay deposits encountered below the Fill and Bay Mud.
- Consolidation tests on Old Bay Clay. The purpose of these tests was primarily to evaluate the stress history (on the basis of maximum past pressure) and to investigate the compressibility characteristics of the Old Bay Clay.
- Unconfined compression tests, point load, and total density tests on rock samples. The purpose of these tests was to evaluate the strength characteristics of specimens of Franciscan Rock.

- Unconsolidated undrained triaxial tests on weathered shale and clayey sandstone samples. The purpose of these tests was to evaluate the undrained strength of the weathered clayey rock.

Detailed testing procedures and results are presented in Appendix B. In the following sections, the results of laboratory tests are summarized and discussed.

5.3.2 Results of Laboratory Tests on Soil

Index Tests

A large number of index tests, including natural moisture content, total density, Atterberg limits, and sieve analysis, were performed for physical characterization and classification.

Moisture contests and total densities in the Fill are plotted against depth on Plate 5-6. In addition to data from this investigation, data collected from the MUNI Metro Turnback Project are included on Plate 5-6. There is a wide scatter in the results, reflecting the heterogeneous nature of the Fill. Moisture contents vary generally from 5 to 30 percent and total densities from 110 to about 138 pcf. Most of the data scatter occurs between the ground surface and a depth of about 10 feet. Below a depth of 10 feet data are more uniform.

There was very little testing on the Bay Mud because, with the exception of borings B-4 and B-11, the boreholes did not encounter this layer. Fifteen feet of Bay Mud were encountered at B-11 and only a thin layer at B-6. A very significant body of information is available from other projects in the area. Moisture contents and total densities versus depth that should be applicable to this area as well, are plotted on Plate 5-7.

Plate 5-8 presents moisture content and total density data for sands, clayey sands, and clays encountered below the Fill and Bay Mud. Less data scatter is observed in these strata than in Fill. Total density ranges from about 120 to 135 pcf, with an average value of 130 pcf. Natural moisture content data are very consistent with depth, with an average value of about 20 percent.

A grain size distribution envelope of sandy samples retrieved from the Upper Sands with Clay Lenses is presented on Plate 5-9. The envelope indicates that the materials are fairly uniform sands with some silt and medium to coarse sand. The percentage of fines ranges from 5 to about 30 percent. Plates 5-10 and 5-11 show grain size distribution envelopes of sandy and clayey samples taken from Alluvial Deposits (from samples taken in borings along the east-west alignments). The

envelope for sandy soils indicate that the material is a fairly uniform fine sand with silt and trace of coarse sand. The percentage of fines varies from 16 to 42 percent. The envelope for clayey samples indicates that the clayey soils contain various amounts of medium to fine sand and traces of fine gravel. Fines content ranges from 53 to 58 percent.

Consolidated Drained Direct Shear Strength Tests

A total of nine consolidated drained direct shear tests was conducted on sandy samples of the Fill, Upper Sands with Clay Lenses, and Older Alluvial Deposits to evaluate the strength and deformation characteristics of these strata. A summary of the test results is presented in Table 5-2, which shows, for each test, fines content, applied normal stress, shear stress at failure as well as the interpreted cohesion and friction angles. Tests on samples of the Older Alluvial Deposit indicate strength parameters that range from 550 psf cohesion intercept and 27 degrees friction angle to 600 psf cohesion intercept and 35 degrees friction angle. For the Upper Sand samples, the strength parameters range from no cohesion intercept and 41 degrees friction angle to 650 psf cohesion intercept and 38 degrees friction. It should be noted that direct shear tests on materials that have significant amounts of fines tend to overestimate the cohesion intercept. Further interpretation of the data and recommendations for design parameters are presented in Section 6.

Unconsolidated Undrained Triaxial Compression Tests

A small number of unconsolidated undrained triaxial compression (UUTC) tests were performed on cohesive samples of Bay Mud, Old Bay Clay and stiff clays to evaluate the undrained shear strength of these strata. Tests were performed at an applied confining pressure approximately equal to the total overburden pressure acting at the depth where samples were obtained. Results of the UUTC tests on cohesive soil samples are presented in Appendix B.

Consolidation Tests

Two consolidation tests were carried out on samples obtained from Old Bay Clay deposits to evaluate the stress history and compressibility of the deposit. Samples were collected from boring B-5 (on Beale Street) at a depth of 100 to 108 feet below ground surface. A standard incremental loading technique was used for the tests in general accordance with Lambe (1961) and ASTM D2435. Since the primary purpose of the tests was to estimate the maximum past pressure and the stress history of the Old Bay Clay, a reduced load increment ratio was used to better define the maximum past pressure in the tests, as described in Appendix B. Compression curves from the consolidation tests are also presented in Appendix B. The results of the tests indicate that the Old

Bay is overconsolidated, with maximum past pressures of 17 to 18 ksf indicating an overconsolidation ratio of about 2. This is consistent with our experience from tests performed at other sites in the downtown area.

5.3.3 Results of Laboratory Tests on Rock

A number of strength tests were performed on selected rock samples to evaluate the strength of intact specimens of Franciscan rock. Unconfined compression tests and point load tests were performed by Advanced Terra Testing, of Lakewood, Colorado, on selected rock core specimens. Rock cores were carefully examined and samples selected for testing so that they had no visible fractures and had minimum dimensions as required for testing. In addition, UUTC tests were performed on softer (weathered) rock samples retrieved with a Pitcher sampler. The Pitcher sampler was used where the rock was too soft to be cored effectively. Pitcher samples were taken from intensely weathered and fractured shale in borings B-3 and B-4 and from clayey sandstone in boring B-8.

Unconfined Compression Tests

A total of 16 unconfined compression tests were performed on rock core specimens. Plate 5-12 shows the variation of unconfined compressive strength with depth below ground surface for sandstone and shale specimens. The compressive strength ranges from less than 200 psi (pounds per square inch) to about 27,000 psi. The wide range and variability of compressive strength observed in the test specimens is the result of the highly heterogeneous character of the Franciscan rock.

Point Load Tests

Point load index tests were performed on 27 rock specimens of Franciscan sandstone and shale. As an index test, the point load test is an inexpensive way to measure the strength of rock specimens without the specimen preparation and setup required for the unconfined compression test. Point load tests were performed on sections of rock core having a length-to-diameter ratio of about 1.0, with appropriate corrections made to account for size effect. In the point load test rock specimens are loaded between steel cones and the load at the point of rupture recorded. The point load strength, I_s, is given by the expression:

$$I_s = C\frac{P}{D^2} \qquad (5\text{-}1)$$

where,

P is the load at rupture,
D is the distance between the point loads at the start of the test, and
C is a specimen size correction factor.

Results of the point load test are presented on Plate 5-13 for both sandstone and shale specimens. Point load index strengths range from 6 psi to about 750 psi, reflecting the high degree of variability of the Franciscan rocks. An attempt was made to correlate the results of unconfined compression tests with those of point load tests Based on 14 pairs of data (from specimens that were used for both tests), the average ratio between the unconfined compressive strength and the point load strength was found to be 29.4, with a standard deviation of 13.2.

Unconsolidated Undrained Triaxial Compression Tests

Eight UUTC tests were performed on clayey rock samples obtained with a Pitcher type sampler. This sampler was used instead of the normal NQ split core barrel in portions of borings B-3 and B-8 where the quality of the rock was not suitable for coring. Highly fractured and weathered shale specimens were recovered from boring B-3 between depths of 58 and 82 feet below ground surface. The rock from boring B-8 was characterized as a highly weathered clayey sandstone with abundant shear zones. Pitcher samples were obtained from depths of 22 to 42.5 feet below ground surface.

Tests were conducted under an all-around confining pressure approximately equal to the total overburden pressure acting at the depth were samples were obtained. The results of the tests are summarized in Table 5-3.

TABLE 5-1
SUMMARY OF FIELD VANE SHEAR TESTS

Field Vane Number	Test Depth (ft)	In-Situ Shear Strength (psf)	Remolded Shear Strength (psf)	Sensitivity
FV-2	24.0	940	-	
	27.5	1129	230	4.9
	31.0	1024	-	
FV-3	31.0	961	-	
	35.0	1003	42	23.9
	39.0	1011	-	
	43.0	1024	167	6.1
	47.0	1003	63	15.9
	51.0	1150	-	
	55.0	1129	376	3.0
	59.0	1317	-	
FV-4	22.5	710	84	8.5
	26.5	752	53	14.2
	30.5	684		
	34.5	773	53	14.6
	38.5	752		
	42.5	1024	63	16.3
	46.5	815		
	50.5	846	63	13.4
	54.5	919		
	58.5	1003	167	6.0
	62.5	982		
	66.5	1065	230	4.6
	70.5	1065		
	74.5	1065	251	4.2
	78.5	1149		
	82.5	1191	396	3.2
FV-5	23.5	731		
	27.5	742	84	8.8
	31.5	742		
	36.0	742	90	8.2
	41.0	951	157	6.1
	45.0	961		
	50.0	815	157	5.2
	54.0	773		
	58.0	877	157	5.9
	63.0	982		
	68.0	982	240	4.1
	85.5	1211	293	4.1
FV-6	15.5	481		
	19.5	460	52	8.9
	23.5	522		
	27.5	627	84	7.5
	31.5	616		
	36.0	888	125	7.1
	40.0	794		

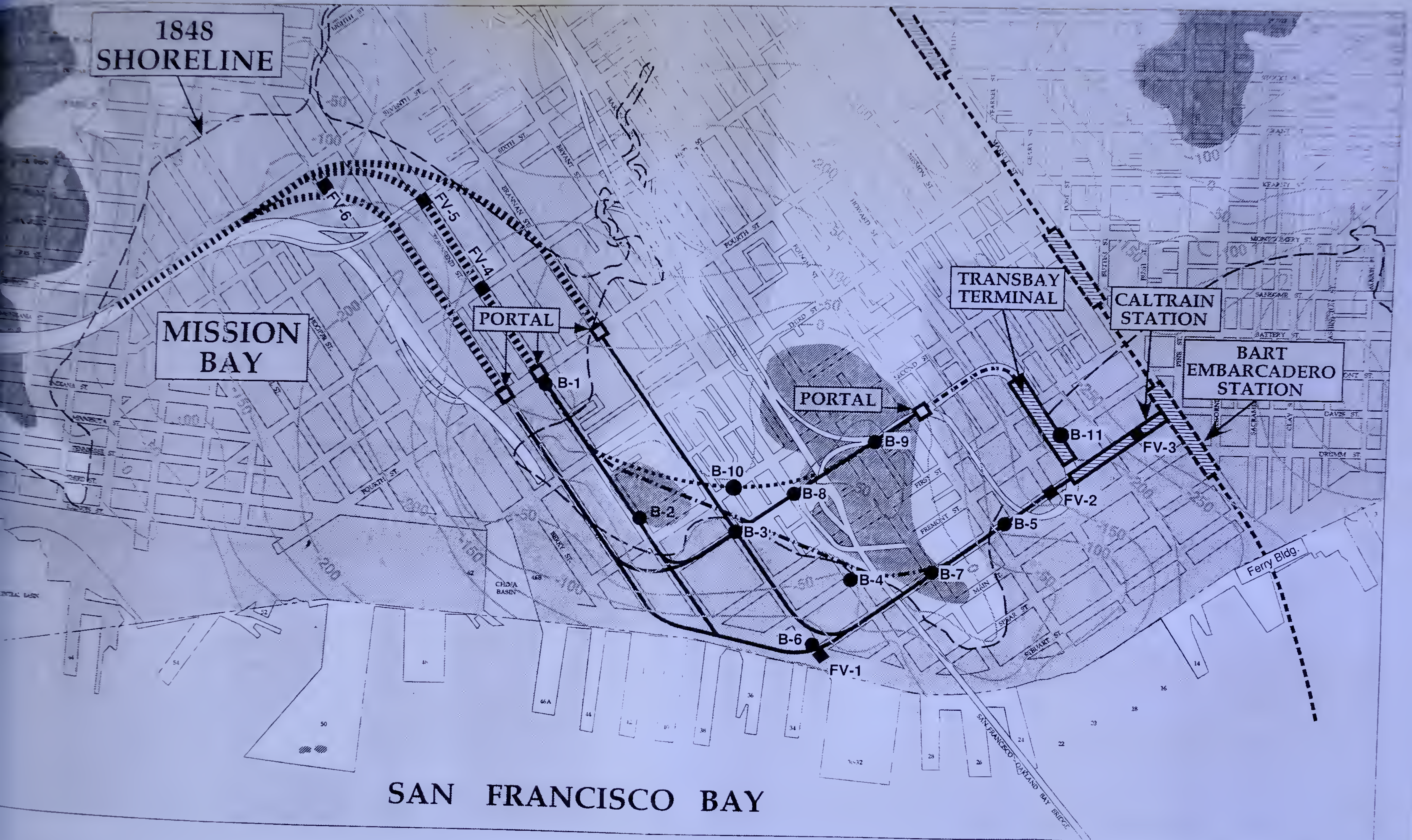
TABLE 5-2
SUMMARY OF CONSOLIDATED DRAINED DIRECT SHEAR TESTS

Soil Unit	Boring No.	Sample Depth (ft)	Soil Type	Fines Content (%)	Normal Stress (psf)	Maximum Shear Stress (psf)	Cohesion Intercept (psf)	Friction Angle (deg)
Fill	B-5	16	SP	1.3	1000 1500 2000	883 1193 1662	150	38
Older Alluvial Deposits	B-1	30	SM/SC	23.7	1500 2500 3000	1719 2351 2808	600	35
	B-1	45	SM	9.7	2000 3000 4000	1941 2683 3353	550	35
	B-8	15	SM/SC	16.8	1000 2000	1064 1584	550	27
Upper Sands with Clay Lenses	B-4	30	SM	7.2	1500 3000	1728 3685		
	B-5	25	SP	4.2	1500 2500 3000	1349 2169 2905	0	43
	B-5	31.5	SM/SC	16.0	1500 2500 3000	1130 1618 1202	200	28
	B-5	45	SP/SM	6.0	2000 3000 4000	2179 2993 3722	650	38
	B-11	56	SP/SM	10.9	2500 4000 5000	2542 3240 4423	0	41

TABLE 5-3
SUMMARY OF UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS ON ROCK SAMPLES

Boring Number	Sample No.	Sample Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Confining Pressure (psf)	Shear Strength (psf)
B-3	8	60	Weathered shale	6.4	147.2	7200	4641
B-3	10	65	Weathered shale	5.8	151.3	7850	3785
B-3	12	69.3	Weathered shale	7.6	149.4	8400	1429*
B-3	14	74.3	Weathered shale	7.5	148.2	9000	2250
B-8	6	26.5	Weathered clayey sandstone	9.0	138.0	3000	2178
B-8	7	29	Weathered clayey sandstone	8.7	140.1	3500	4132
B-8	10	36.5	Weathered clayey sandstone	10.6	136.8	4450	4035
B-8	11	39	Weathered clayey sandstone	8.9	139.2	4800	3603

*Failure along discontinuity.



KEY

● B-5 Boring Location

■ FV-3 Field Vane Location

Bedrock Contours (feet)

Fill

▨ Rock Outcrop

PLOT PLAN SHOWING SOIL BORINGS DRILLED FOR THIS INVESTIGATION AND PROPOSED CALTRAIN ALIGNMENTS

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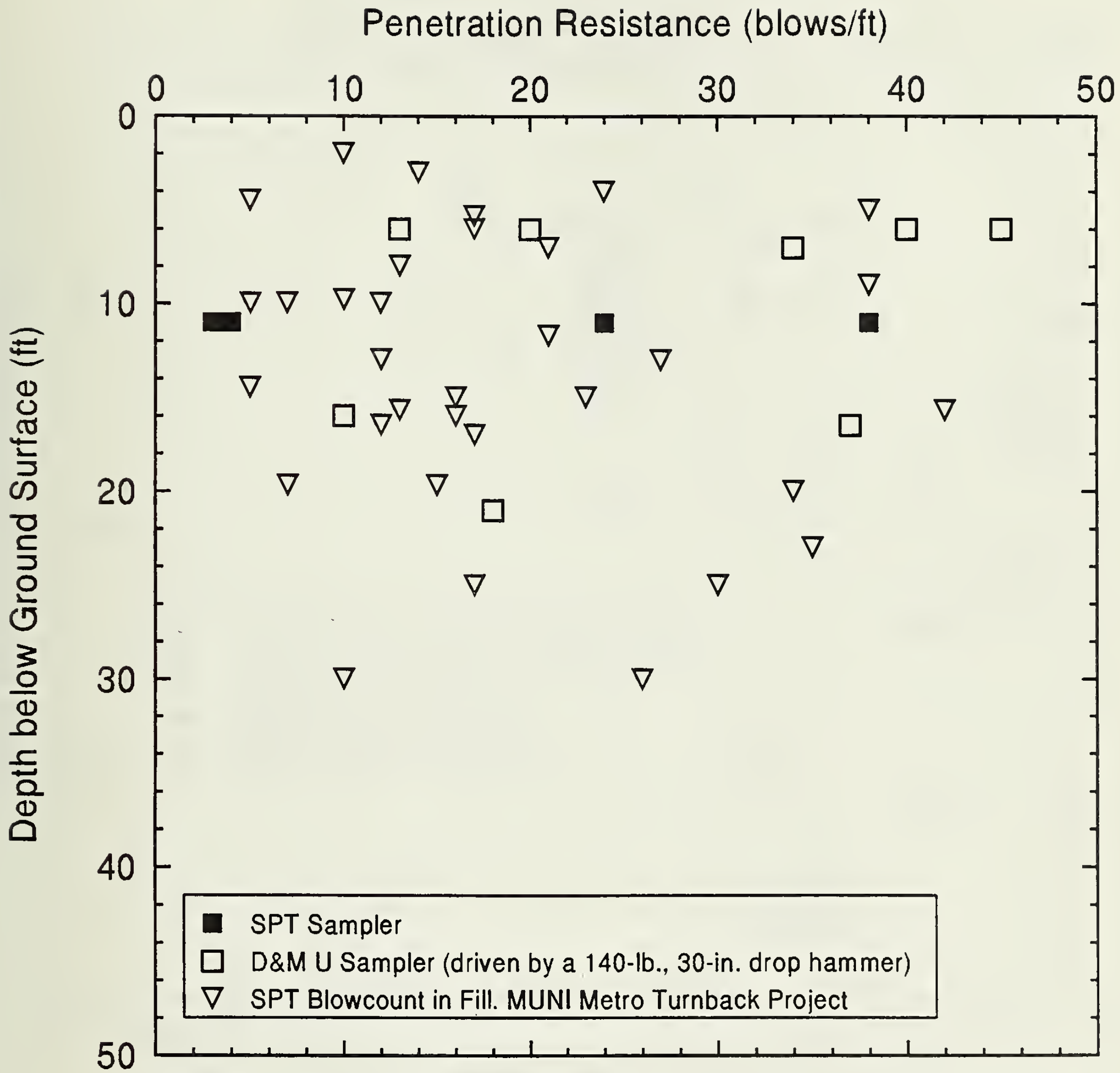
PLATE 5-2



Note:

To convert D&M U values to SPT values multiply by 0.38

PENETRATION RESISTANCE IN FILL

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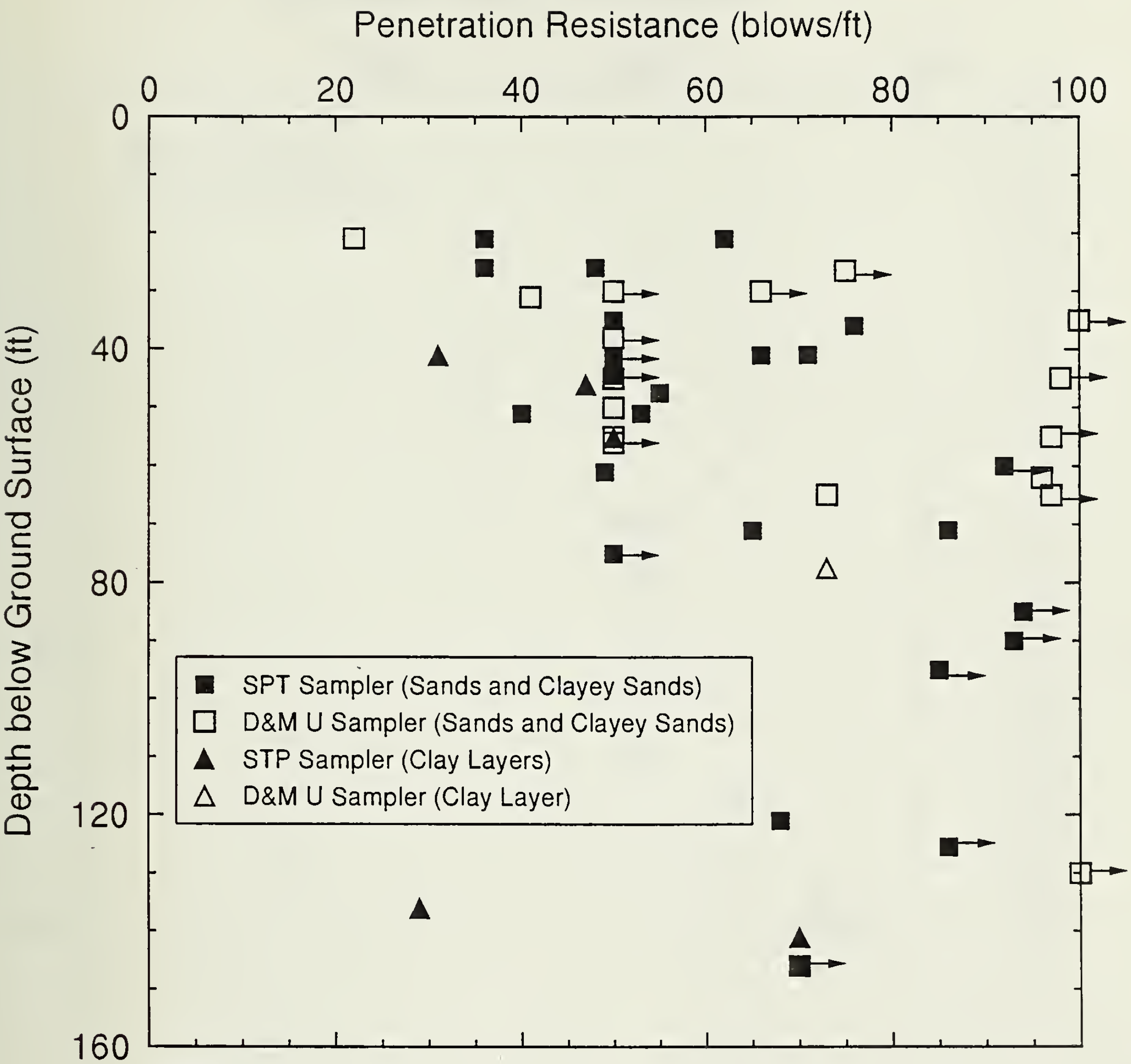
PLATE 5-3



Notes

(1) → Indicates Blow Counts for Less Than One Foot of Penetration

(2) D&M U-Sampler driven by a 140-lb., 30-in. drop hammer.
(to convert U values to SPT values multiply 0.38)

PENETRATION RESISTANCE IN SANDS, CLAYEY SANDS, AND CLAYS BELOW FILL AND BAY MUD
(Combined Upper Sand and Lower Sand Strata)

September 1995
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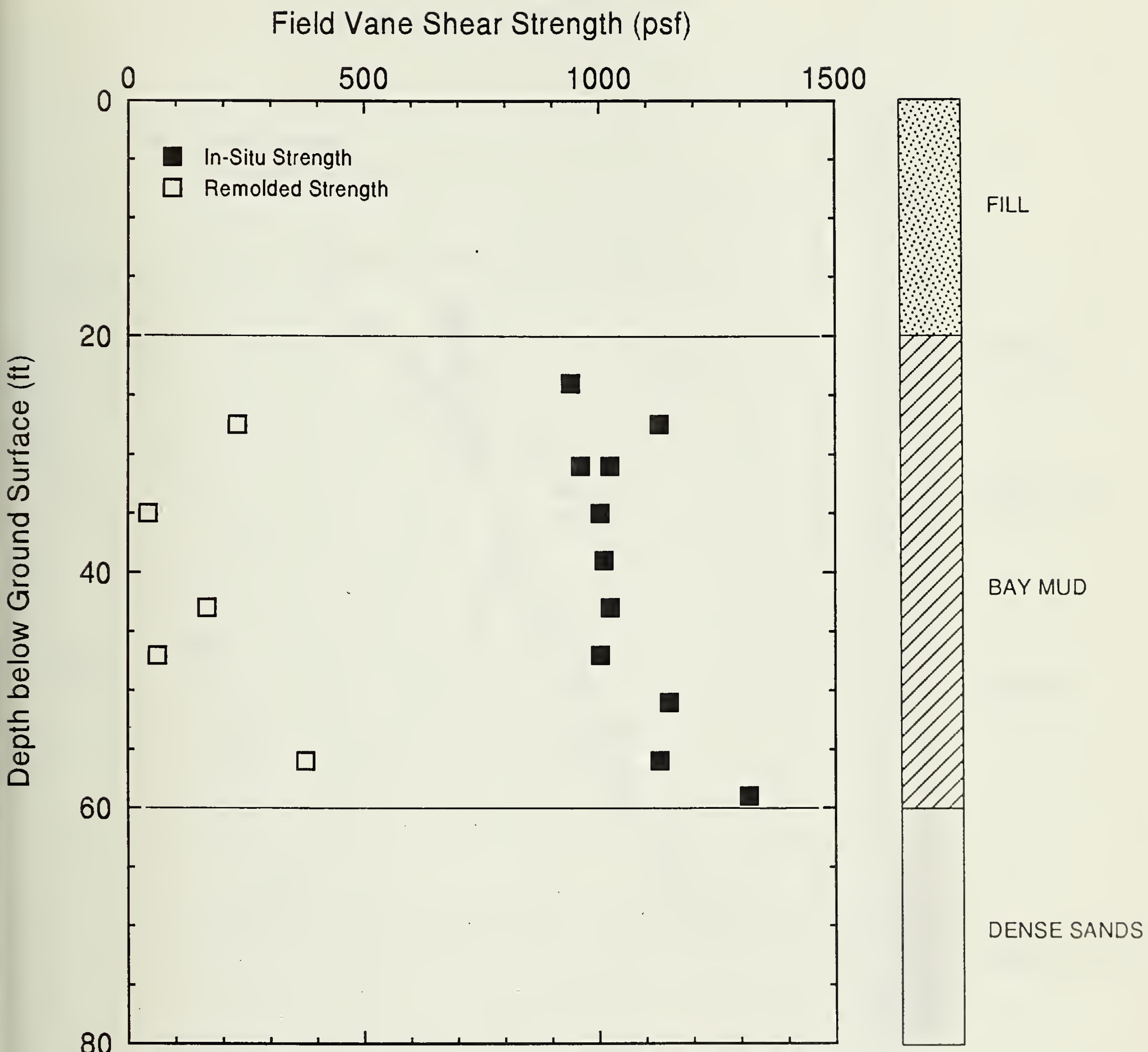
PLATE 5-4



RESULTS OF FIELD VANE TESTS FV-2 AND FV-3
FORMER YERBA BUENA COVE

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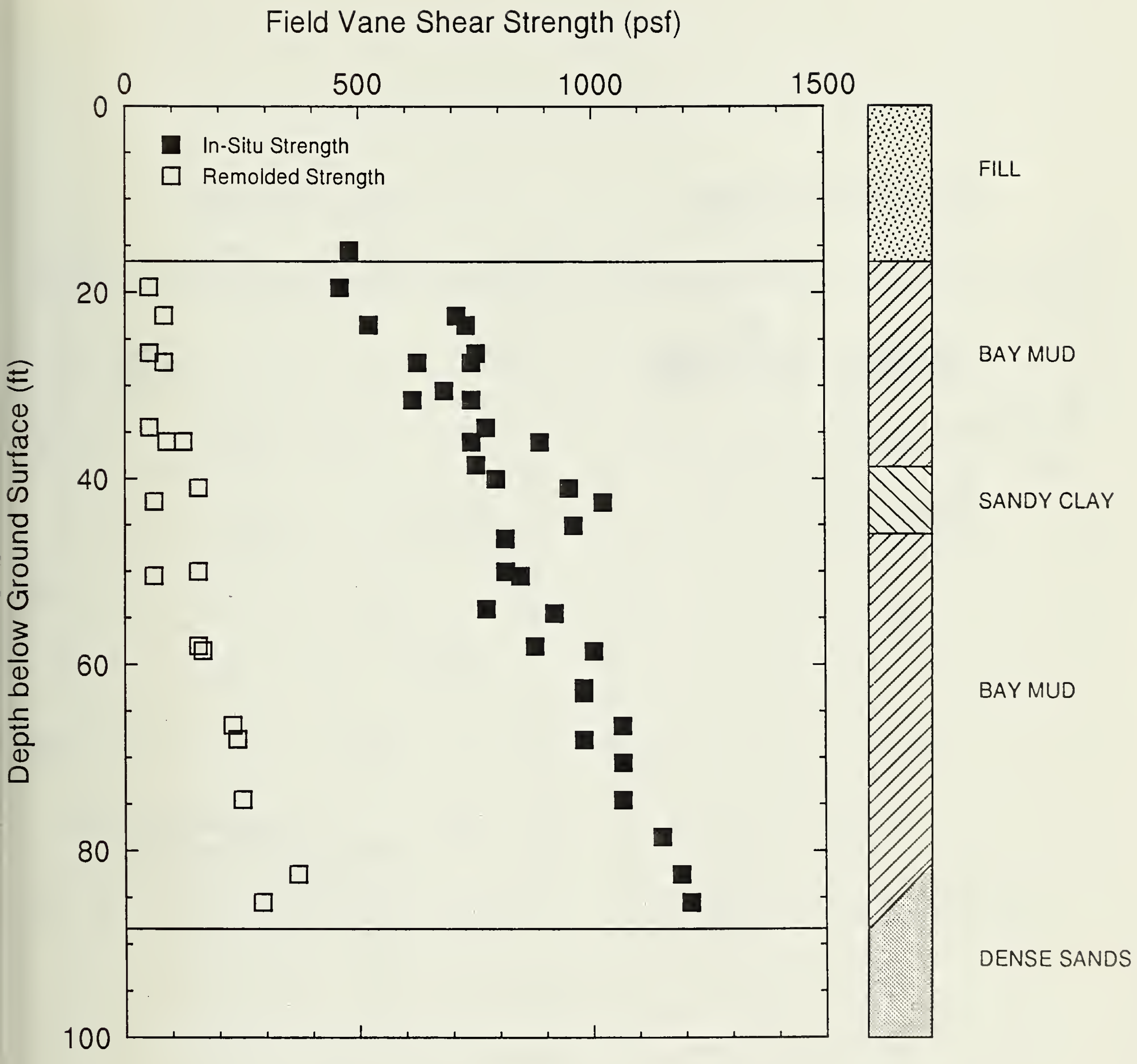
PLATE 5-5A



RESULTS OF FIELD VANE TESTS FV-4, FV-5, AND FV-6 FORMER MISSION BAY

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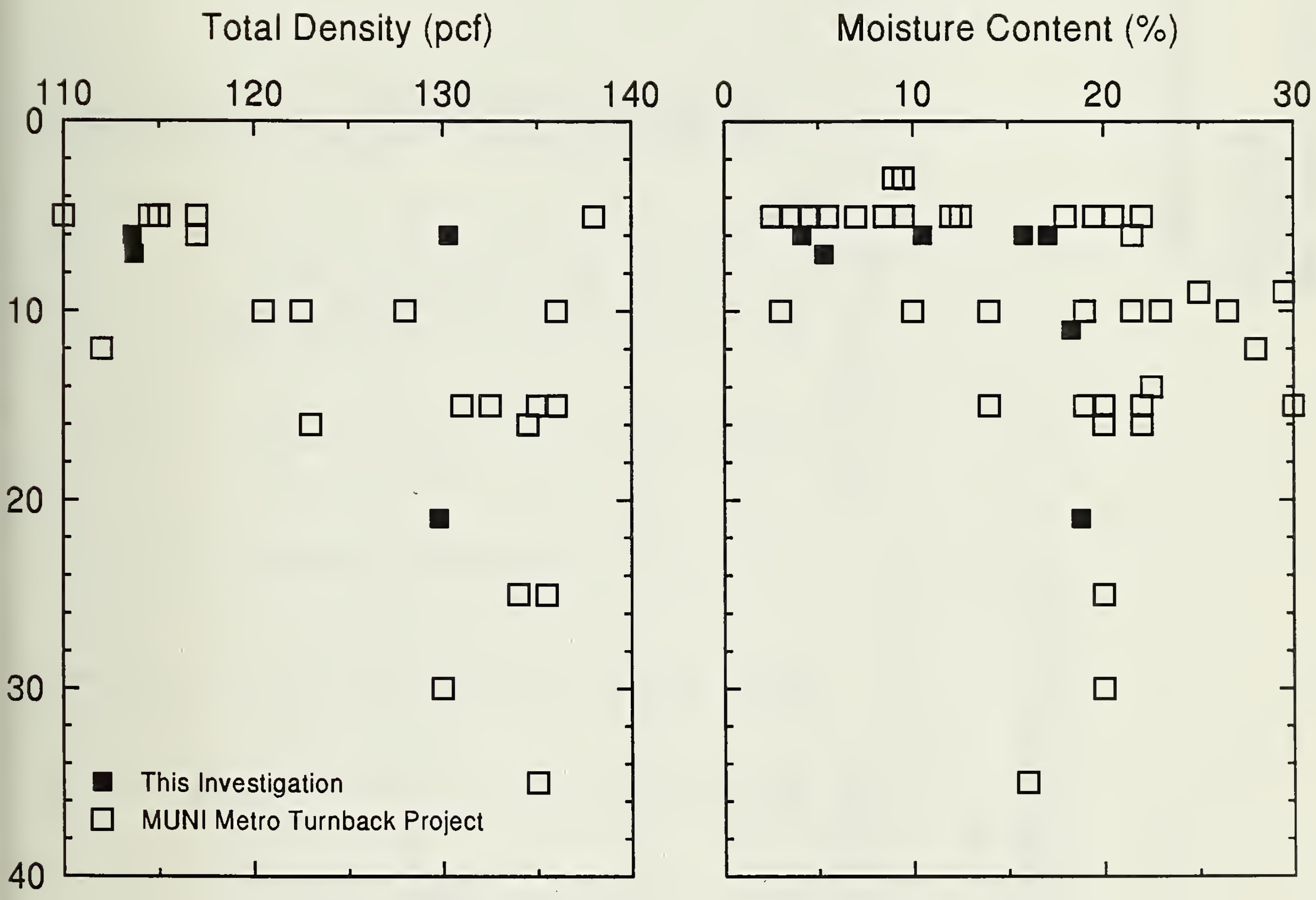

TOTAL DENSITY AND NATURAL MOISTURE CONTENT FOR FILL

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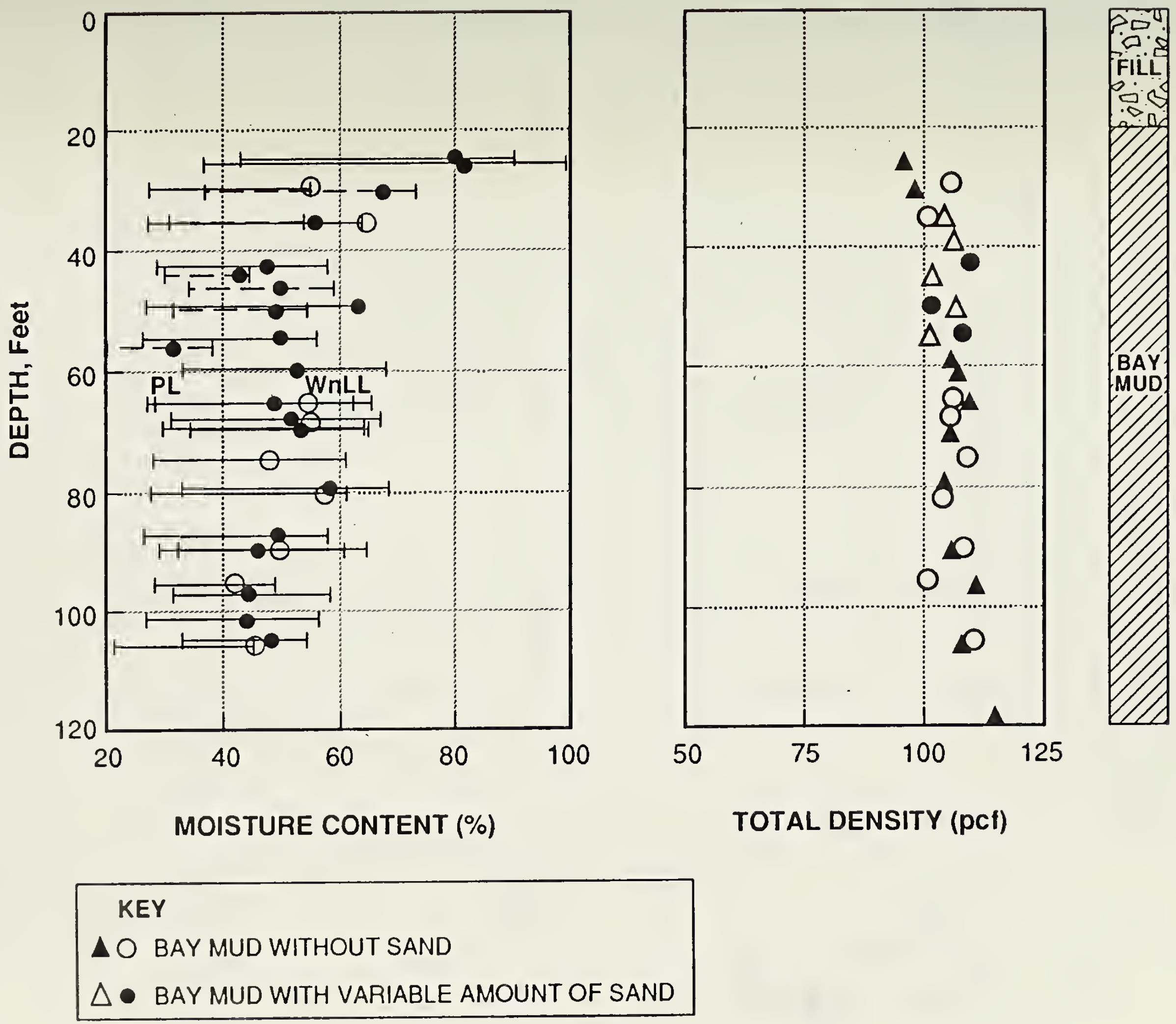
PLATE 5-6



INDEX PROPERTIES OF THE RECENT BAY MUD

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Relocation EIS/EIR Project

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30751-001-003

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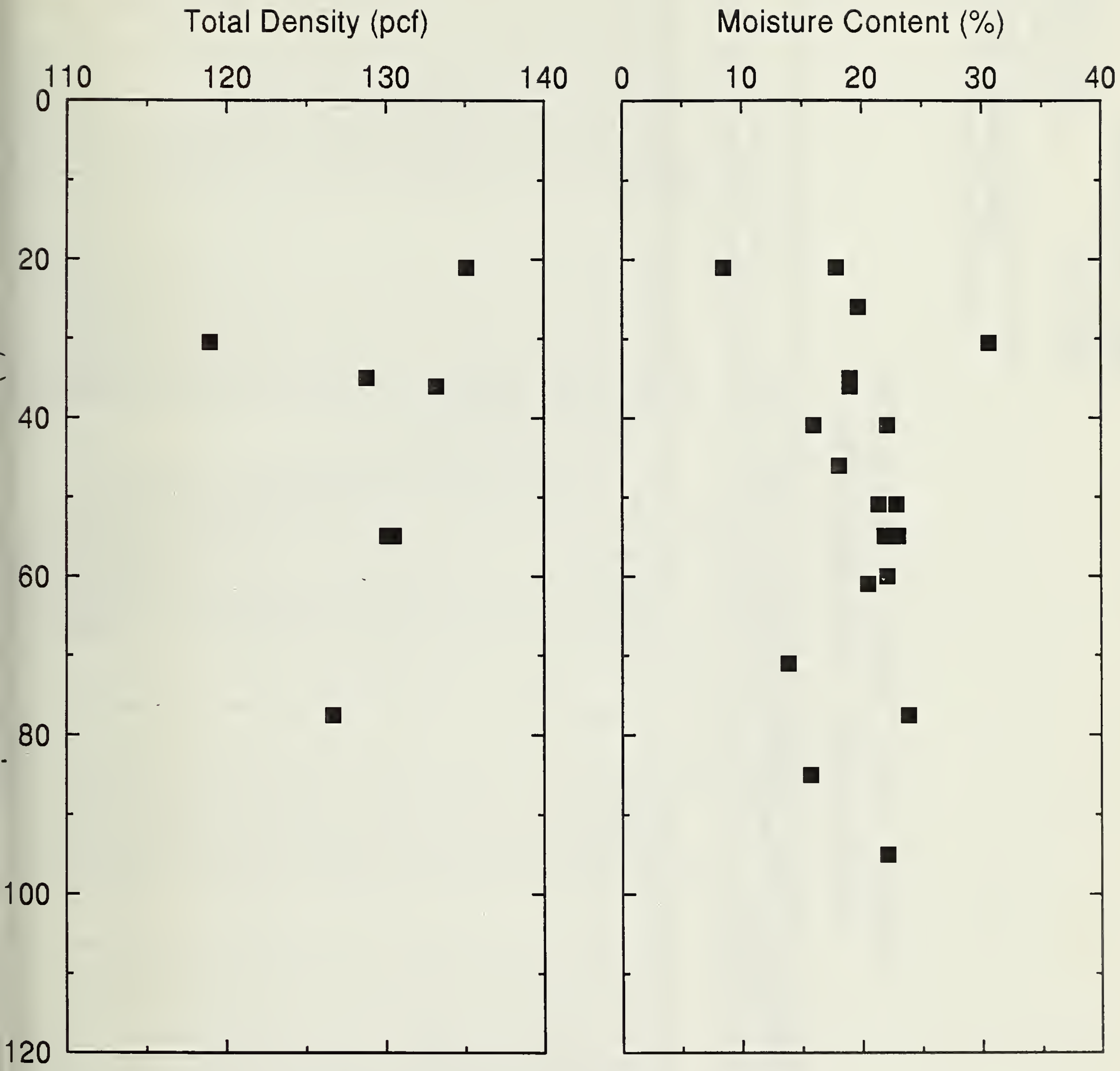
PLATE 5-7



TOTAL DENSITY AND NATURAL MOISTURE CONTENT FOR SANDS, CLAYEY SANDS, AND CLAYS BELOW FILL AND BAY MUD

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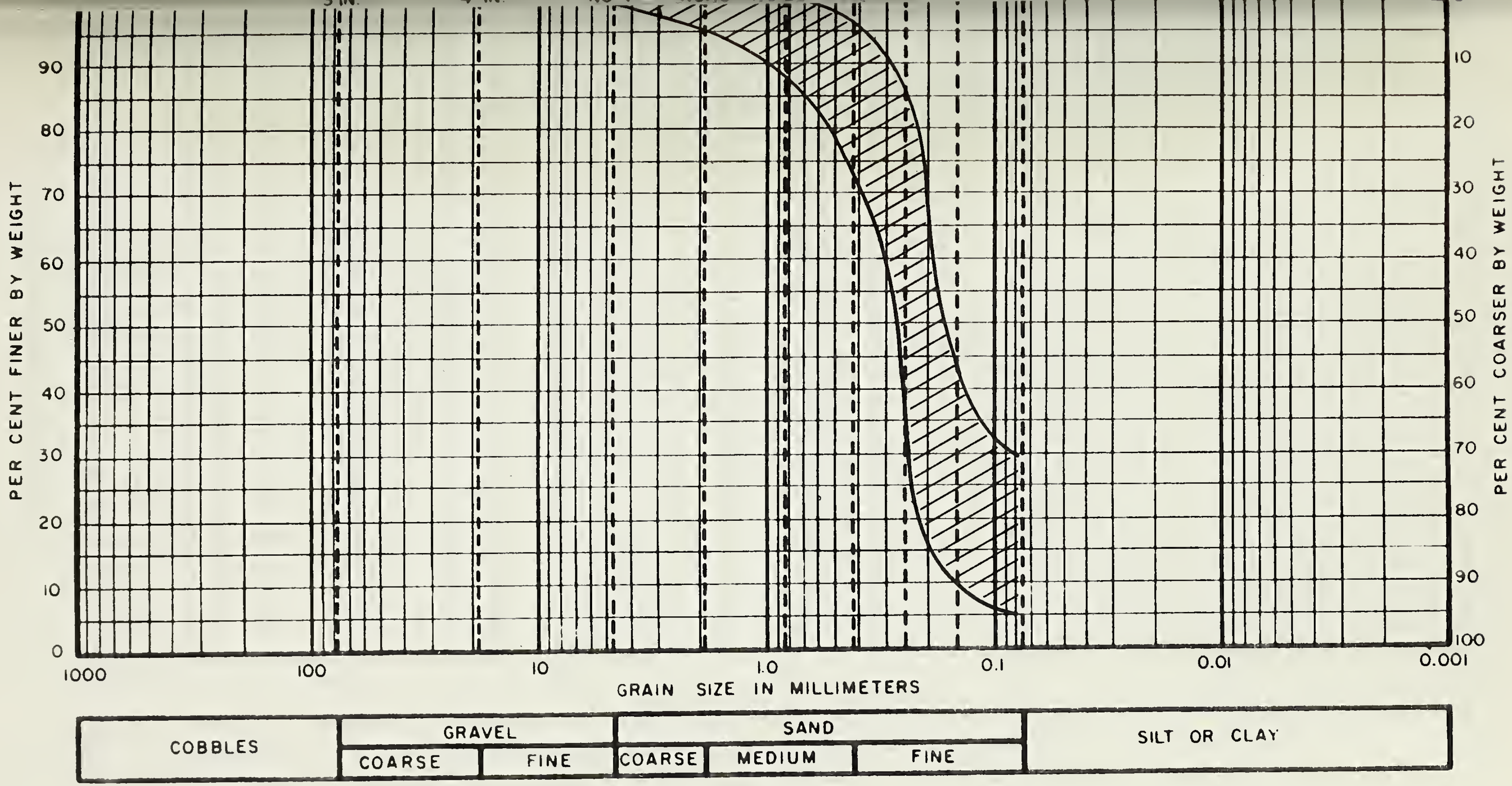
PLATE 5-8



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

GRAIN SIZE DISTRIBUTION ENVELOPE FOR UPPER SANDS WITH CLAY LENSES

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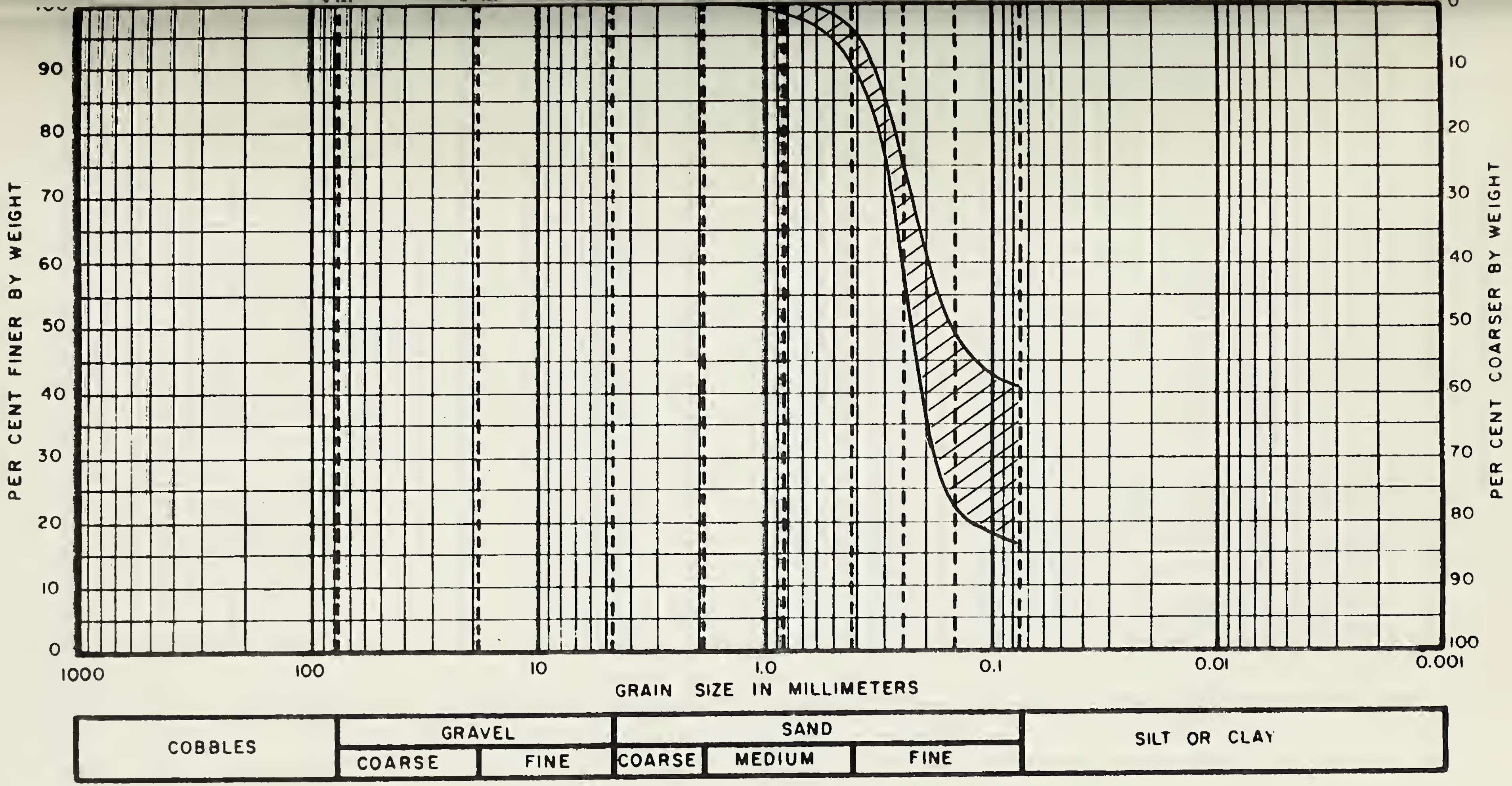
PLATE 5-9



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

GRAIN SIZE DISTRIBUTION ENVELOPE FOR SANDY SAMPLES. OLD ALLUVIAL DEPOSITS

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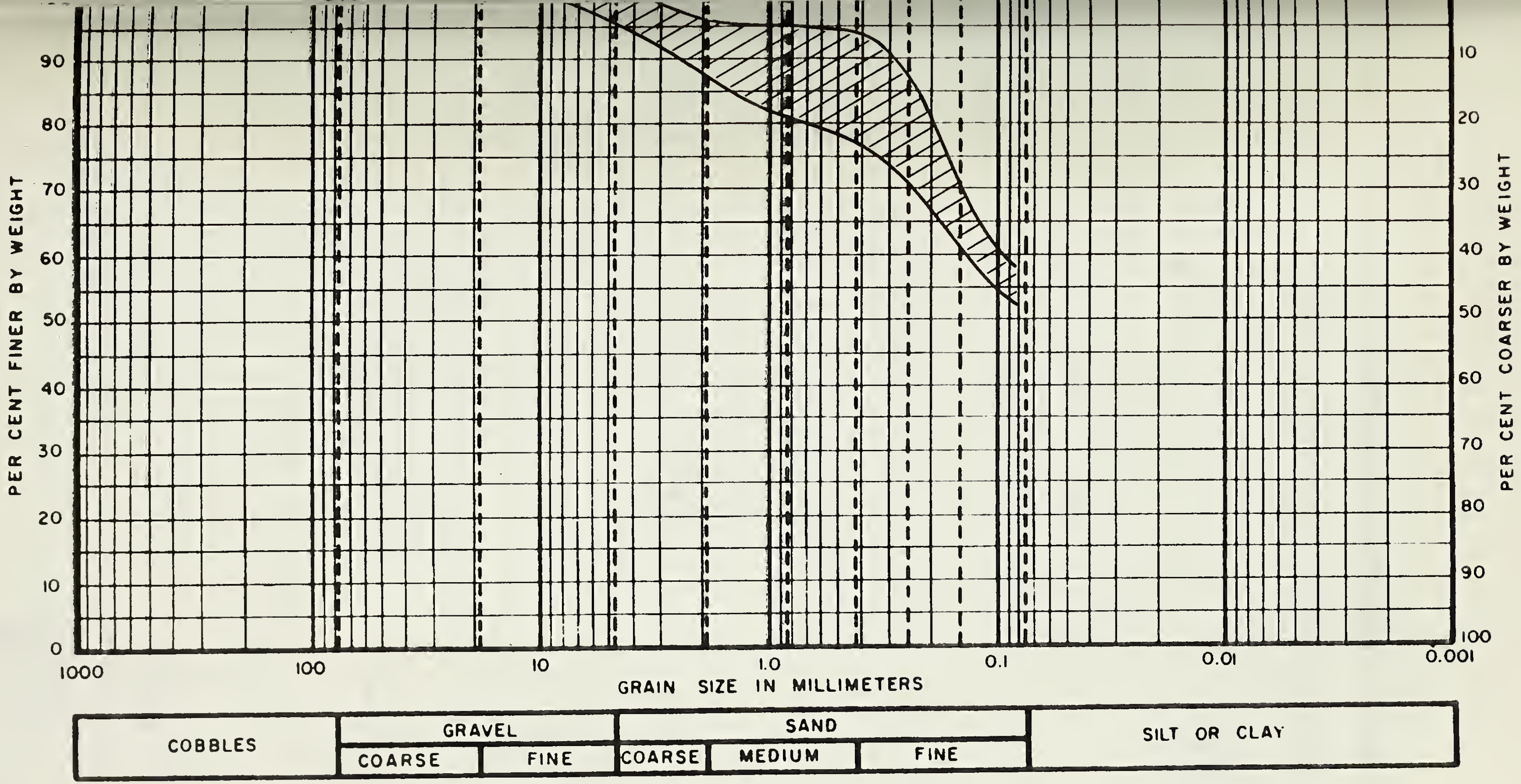
PLATE 5-10



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

GRAIN SIZE DISTRIBUTION ENVELOPE FOR CLAYEY SAMPLES. OLD ALLUVIAL DEPOSITS

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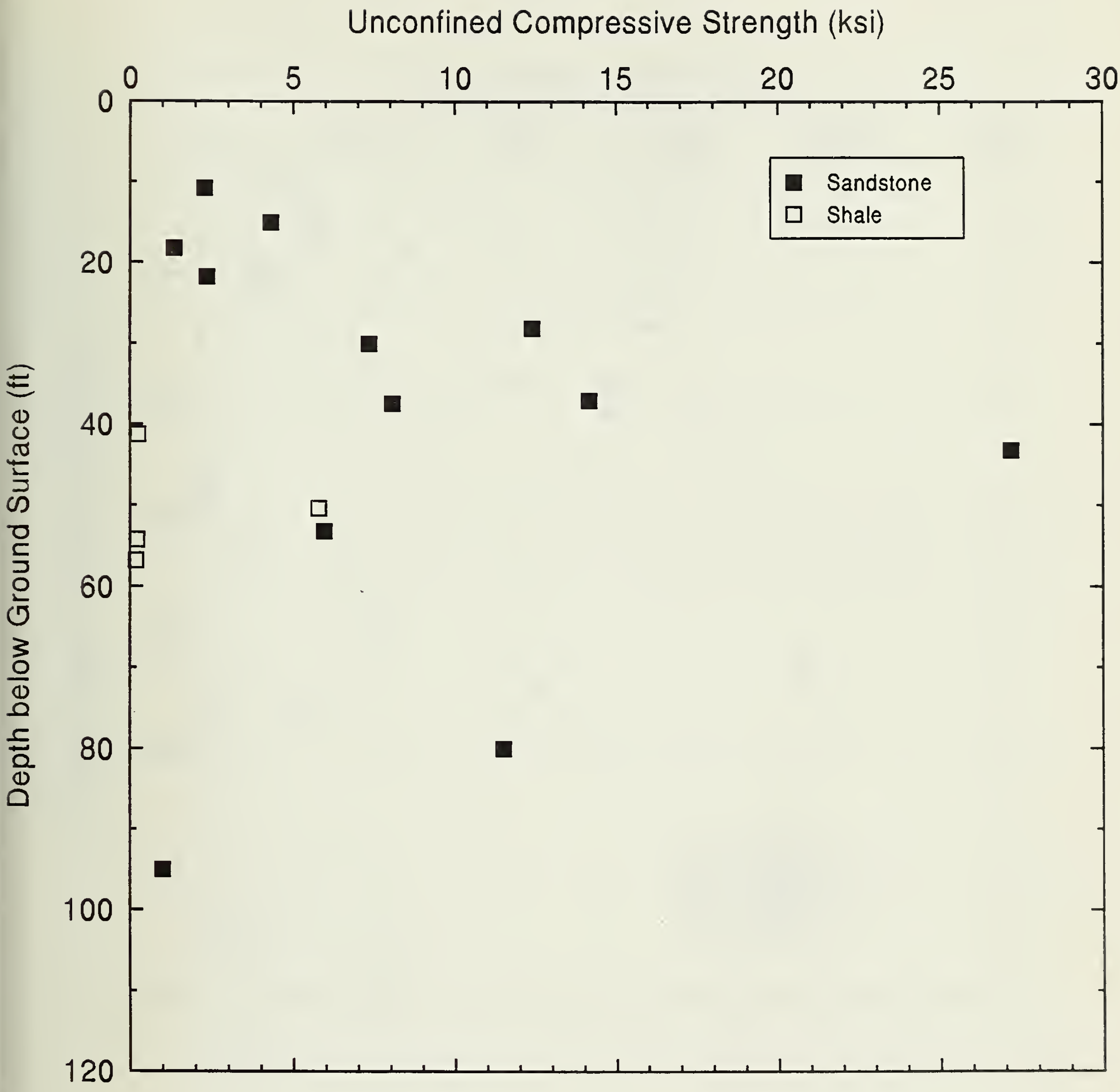
PLATE 5-11



RESULTS OF UNCONFINED COMPRESSION TESTS ON ROCK SAMPLES

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Caltrain S.F. Downtown Station
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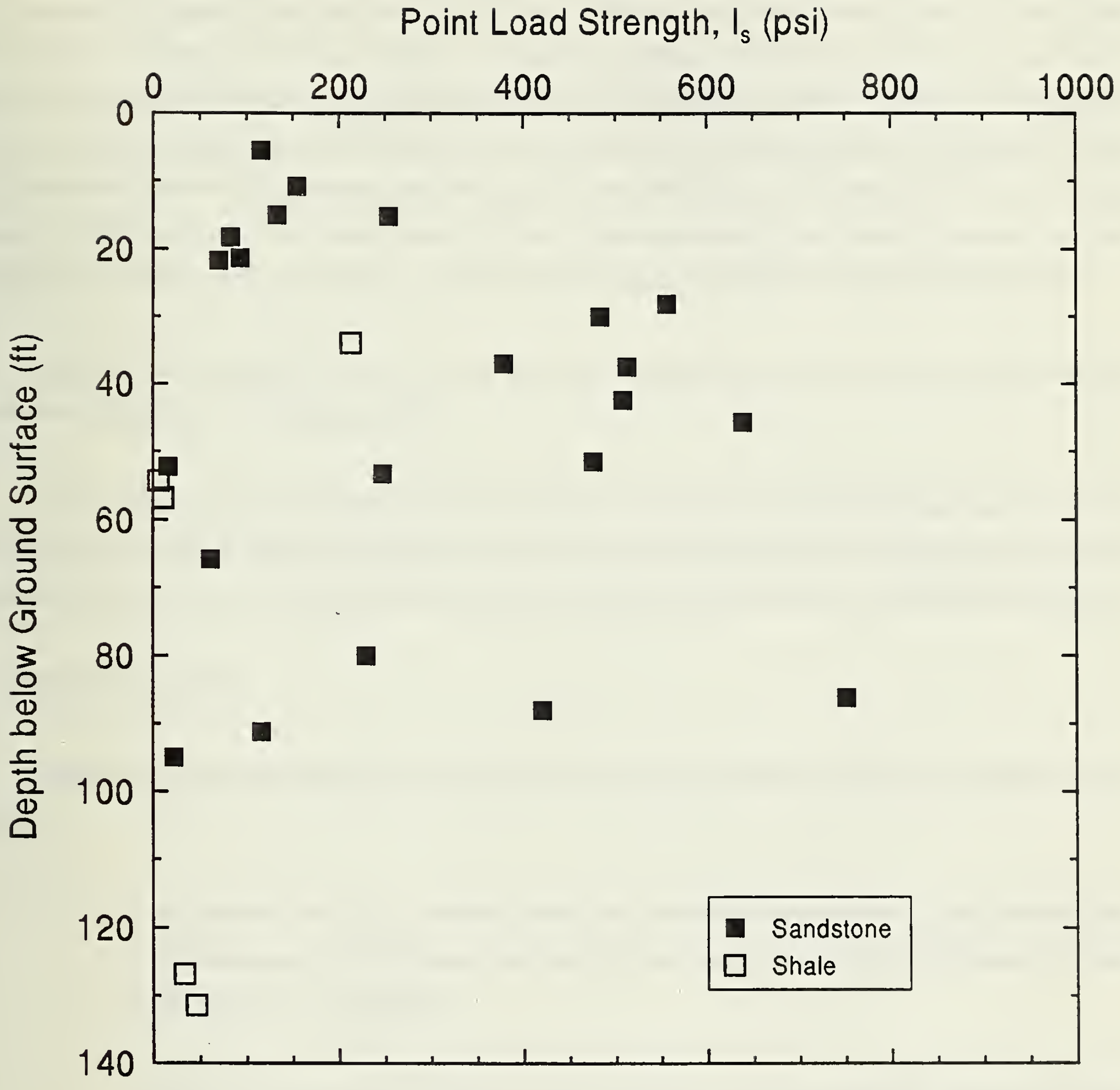
PLATE 5-12



RESULTS OF POINT LOAD TESTS ON ROCK SAMPLES

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6.0 SUBSURFACE CONDITIONS AND PARAMETERS FOR ANALYSIS AND DESIGN

Plate 6-1 shows the study corridor superimposed on the U.S. Coast Survey Map of 1856, which shows the original (1848) shoreline and the boundaries of marshlands and creeks in the vicinity of the project. As can be seen from Plate 6-1,a significant portion of Alternative 2A-C (cut-and-cover alternative) lies within the area reclaimed from the old Yerba Buena Cove. Portions of Alternative 2A-C also pass through the South Beach Area, outboard of the old shoreline. The Brannan Street and Townsend East-West alignments pass through the area just inland of Steamboat Point and extend further into Mission Bay. The major portion of the alignment along King Street is located outboard of the old shoreline, while the major portion of Alternative 3 lies inboard of the old shoreline.

The subsurface conditions vary to a large degree, depending on the relative position of each alternative alignment to the old shoreline.

The general geology and sequence of stratification of the site were briefly discussed in Section 4. This section provides detailed description of the major strata encountered within the limits of the proposed alignment and recommends appropriate engineering parameters for analysis and design.

6.1 STRATIGRAPHY

Subsurface sections have been prepared for the various portions of the study alternatives as follows:

1. For Alternative 2A-C, sections were prepared for the portions along Beale Street from Mission to the Embarcadero; and for the portions along the Embarcadero from Beale Street to King Street.

2. Along the east-west direction cross sections were prepared along Brannan, Townsend, and King streets, extending from the Embarcadero to Fourth Street.

3. Along portions of Alignment 2A′ from the intersection of Beale and Harrison streets at the north end to the intersection of Third and Townsend streets to the south.

4. For Alignment 3A-3B, two cross sections were prepared. The first one was from Mission Street at the north end to Townsend Street at the south, running approximately along the alignment of Colin P. Kelly Jr. Street. The second one was

constructed along the proposed tunnel from Harrison Street to approximately Third and Townsend streets.

5. Along Townsend Street from Fourth Street to near Seventh and Berry streets.

6. For the Transbay Transit Terminal, a subsurface cross section was prepared running through the terminal in the east-west direction from a point approximately 500 feet west of First Street to a point about 100 feet east of Main Street.

The subsurface sections are shown on Plates 6-2 to 6-11. The major strata shown on the various sections are described below.

- At the surface there is a layer of Fill of variable thickness and composition. It ranges from brown to gray and tan fine Sand to black fine Clayey Sand with frequent rubble, wood, concrete and brick fragments. It varies in thickness from less than 5 feet to as much as 30 feet. It is typically very loose to medium dense.

- Below the Fill is a layer of medium stiff to stiff fat clay known locally as Bay Mud. Along the Beale Street portion of Alternative 2A-C, Bay Mud is present between Market Street at the north end and a point about 200 feet south of Howard Street. Bay Mud is also present between Bryant Street and the Embarcadero, while it is absent over most of the portion of the alignment between Folsom and Bryant streets. At the north end of the Beale Street portion of Alternative 2A-C, the Bay Mud is as much as 45 feet thick, but its thickness decreases gradually from north to south until a point about 200 feet south of Howard Street where it pinches out.

  South of Bryant Street the thickness of the Bay Mud ranges from 7 feet to 20 feet, with its thickness increasing gradually from north to south.

  The Bay Mud is typically present in the areas outboard of the old (1848) shoreline, although Bay Mud is found in some areas inboard of the old shoreline, apparently representing former low lying areas such as marshlands and creek beds that were filled with bay sediments.

- Below the Bay Mud is a system of sand deposits that vary substantially in thickness This system is referred to herein as the "Upper Sands." In some areas outboard of the old shoreline, immediately below the Bay Mud, there is a layer of dark gray loose to

medium dense Clayey Sand. This layer is typically found in the area encompassed by the old Yerba Buena Cove, north of Rincon Point. The presence of this layer south of Rincon Point is sporadic, and where present, it is very thin.

The major sand stratum below the Bay Mud is a dense to very dense, brown and tan medium to fine Sand with variable amounts of silt. This stratum is known locally as the Colma Formation, and it is slightly cemented. The thickness of this deposit varies substantially within the study area. This layer is thickest in the area between Howard and Folsom streets along the Beale Street portion of Alternative 2A-C. The Colma is also fairly thick along the portion of Beale Street between Bryant and the Embarcadero.

- Within the corridor of the east-west portions of the various alternatives running along Brannan, Townsend and King streets, there is a system of dense to very dense sands, interbedded with occasional stiff clay lenses of variable thickness below the Fill (where the Bay Mud is absent) and Bay Mud strata. The dense sands and stiff clays probably constitute a sequence of Colma Sand and older Alluvial or Bay deposits.

- Within the portion of the study corridor that is outboard of the old shoreline (see Plate 6-1), below the Upper Sands is a layer of very stiff, plastic clay known locally as Old Bay Clay. The thickest accumulations of Old Bay Clay are found along the Beale Street portion of Alternative 2A-C. The Old Bay Clay is up to 125 feet thick in the area between Mission and Market streets, but it decreases in thickness from north to south and pinches out near Folsom Street (north of Rincon Hill). It reappears near the intersection of Beale Street with the Embarcadero. Substantial thicknesses of Old Bay are also found in the Mission Bay Area.

  The Old Bay Clay is known to contain relatively thick layers and lenses of dense sands, but the continuity of these dense sand layers is uncertain. The Old Bay Clay is also fairly thick in the area of the Transbay Transit Terminal as can be seen from Plate 6-11. Based on geologic considerations, it would be reasonable to expect that Old Bay Clay could be present also along the portion of the alignment that runs along the Embarcadero; however, the available information (from boreholes drilled previously by others) does not provide conclusive evidence as to the continuity and thickness of the Old Bay Clay. It should be expected that stiff clay layers, probably Old Bay Clay, are present at various depths, with variable thicknesses within the layer

identified on the sections of Old Bay Deposits. Old Bay Clay of variable thickness is also present along the portion of Alternative 3 between Mission and Tehama streets.

- Below the Old Bay Clay is a system of interbedded dense sands and stiff clays, referred to in the sections as Lower Sands with Clay Lenses, or in short, "Lower Sands." These deposits are highly variable in thickness and may be of similar geologic origin as the Old Bay deposits or the older Alluvial deposits indicated on the east-west sections (along Brannan, Townsend, and King streets).

- Based on our experience, and from the results of the boreholes drilled for this investigation, it is expected that a thin layer of Colluvium, typically consisting of stiff Gravelly Clay, may be found just above bedrock. However, the presence of this layer is not consistent throughout the study corridor.

- The sequence of sediments described above, is underlain by bedrock of the Franciscan Formation. It is an assemblage of highly fractured and weathered, interbedded shales and sandstones. The rock is exposed at the surface, in the area of Rincon Hill, while elsewhere the top of the rock may be up to 250 feet below ground surface. The rock is highly variable, ranging from residual soil to sound unweathered rock.

The following sections present detailed descriptions of the major strata and relevant engineering properties for preliminary design purposes.

6.2 FILL

The fill layer ranges in thickness from a few feet up to 30 feet and can be characterized as loose to medium dense, fine to medium sand with variable amount of silt. It contains obstructions, including rubble, concrete, bricks, and wood. For engineering analysis we recommend an average total unit weight of 120 pcf and a friction angle of 32°. This layer is susceptible to liquefaction.

6.3 BAY MUD

The Bay Mud is a compressible fat clay (classified as CH according to the Unified Classification System) ranging from medium stiff near the top of the layer to stiff near the base of the deposit. It contains various amounts of shells, sand and organic matter, particularly near the top of the layer. In the Mission Bay Area, distinct layers of sandy Bay Mud were found at depths of 20 to 30 feet below the top of this stratum.

Undrained Strength Characteristics

The Bay Mud is known to follow normalized behavior with regard to its undrained shear strength characteristics. Furthermore, the Bay Mud is quite anisotropic, which means that the undrained shear strength (Ladd and Foott, 1974; Ladd et al., 1977) varies with the direction of loading. The strength is highest where the soil is sheared by increasing the vertical stress (active condition), and has the least strength when it is sheared by increasing the horizontal stress (passive condition). When it is sheared along a horizontal plane it has intermediate strength. For normally consolidated Bay Mud the following normalized strength ratios are appropriate.

Compression: $S_u/\bar{\sigma}_v = 0.30$

Direct Simple Shear: $S_u/\bar{\sigma}_v = 0.23$

Extension: $S_u/\bar{\sigma}_v = 0.18$

Undrained strengths measured with the field vane, are essentially the same as strengths measured with the direct simple shear test.

The effects of overconsolidation can be accounted for by the following equation.

$$S_u = S \times \bar{\sigma}_{vo} \times (\text{OCR})^{0.85}$$

where S_u is the undrained shear strength.

S	is the appropriate $S_u/\bar{\sigma}_v$ value for normally consolidated soil.
$\bar{\sigma}_v$	is the vertical effective stress
$\bar{\sigma}_{vo}$	is the in-situ effective overburden stress
OCR	is the overconsolidation ratio = $\bar{\sigma}_{vmax}/\bar{\sigma}_{vo}$
$\bar{\sigma}_{vmax}$	is the maximum past pressure from consolidation tests.

The undrained shear strength of the Bay Mud increases with depth. Undrained strengths measured at five field vane shear test locations are summarized on Plates 6-12 to 6-14. The vane shear strength is a reliable index test. Values of vane shear strengths need to be corrected to account for anisotropic and rate effects using empirical correction factors (Bjerrum, 1972; Mesri, 1989). For the Bay Mud a correction factor of 0.9 is typically used. The corrected vane shear strengths are very close to the strengths determined from SHANSEP principles and $S_u/\bar{\sigma}_v$ values determined from direct

simple shear tests. Along the portions of the alignment that traverse the Old Yerba Buena Cove area, the Bay Mud is typically stiff with strengths in the range of 850 to 1,000 psf. For preliminary conceptual design purposes, a shear strength value of 900 psf is recommended.

The results of the vane shear strength tests performed at two locations along Beale Street (in the area of the Yerba Buena Cove) indicate that the Mud is preconsolidated. The preconsolidation is likely to have been induced by construction dewatering and perhaps by pumping from the sand layers below the Mud to withdraw water for commercial purposes in the early 1900s. Notable construction activities that might have induced consolidation of the Mud due to dewatering include construction of the BART system and various high-rise buildings with deep basements.

We have no relevant information for the South Beach area. However, based on our experience, we would anticipate that the mud in this area is normally consolidated, because of the proximity of this area to the Bay and due to the absence of notable construction activities that could have impacted the Bay Mud. The strength of the Bay Mud can be estimated from normalized strength behavior. For preliminary design purposes we recommend strength values estimated from:

$$S_u = 400 + 9\ (d - 20)$$

where d is the depth below ground level in feet, and S_u is expressed in pounds per square foot.

In the Mission Bay Area the Mud may be assumed to be normally consolidated for the most part. However, at the location of vane shear tests FV-4 and FV-5, the top 20 feet of the Mud appear to be lightly overconsolidated with measured vane strengths of about 750 psf. For design purposes the strength of the Mud in the Mission Bay area, at least along Townsend Street, may be assumed to be constant at the top 20 feet (depth interval at 20 feet to 40 feet) at about 650 psf, and increasing linearly with depth below a depth, d, of 40 feet as follows:

$$S_u = 675\text{ psf} + 9\ (d - 40)$$

Other Parameters

For preliminary engineering purposes, we recommend the following parameters for the Bay Mud:

Parameter	Value
Total Unit Weight	100-105 pcf
Coefficient of Earth Pressure at Rest	0.55
Permeability	1×10^{-7} cm/sec

6.4 UPPER SANDS

The upper sands consist of a medium dense dark gray Clayey Sand, of marine deposition, and the rust brown and tan medium to fine Sand, known as Colma.

The Gray Sand is typically less than 10 feet thick and is essentially absent in the area south of Rincon Hill. The Gray Sand is quite silty and clayey and may contain stringers of clay. For engineering analysis a total unit weight of 120 pcf and an effective friction angle of 32°, with no effective cohesion, are recommended.

The Colma sand is typically dense to very dense with standard penetration (N) values in the range of 40 to 80 blows per foot. It is a medium to fine sand with variable amounts of silt, and is slightly cemented. The cementation is highly variable. For preliminary design purposes we recommend a total unit weight of 130 pcf and an effective friction angle of 35° with no effective cohesion.

6.5 OLD BAY CLAY

Extensive deposits of Old Bay Clay are present in the area north of Rincon Hill, in the South Beach area toward the waterfront, and in the Mission Bay Area. The maximum thickness of this layer occurs in the area between Mission and Howard streets, where it is as much as 125 feet thick. The Old Bay Clay is a low to medium plasticity clay classified as CL or CH according to the Unified Soil Classification System. The Old Bay Clay is interbedded with sand lenses of variable thicknesses and horizontal extent. It is not possible to reliably identify the areas and depth of the sand lenses based on the available information. A total unit weight of 115 pcf is recommended for this layer.

The Old Bay Clay is overconsolidated. Numerous consolidation tests performed for various projects in the vicinity of the study corridor as well as tests performed for this study indicate overconsolidation ratios in the range of 1.5 to 2.

The Old Bay Clay is very stiff with undrained shear strength that increases approximately linearly with depth below the top of this layer. For preliminary engineering analysis a strength of 1,500 psf can be assumed at a depth of 80 feet increasing at a rate of 20 psf per foot of additional depth.

6.6 LOWER SANDS WITH CLAY LENSES

The lower sands are highly variable and appear to consist of a sequence of sediments that vary from fairly clean medium to fine sands to clayey sands. Within the lower sands are layers of very stiff clays, similar to the Old Bay Clay; although the deeper clay layers contain trace to little fine gravel, suggesting that these sediments may be of Alluvial origin, at least in the areas inboard of the old shoreline. The strength of these very stiff clays probably exceeds 2,500 psf. A total unit weight of 120 pcf may be used for the clay layers.

The sands are very dense, with standard penetration resistances ranging from 60 to over 100 blows per foot. A total unit weight of 130 pcf is recommended for calculation purposes. An effective friction angle of 35° with no effective cohesion are recommended for analysis.

6.7 OLDER ALLUVIAL AND BAY DEPOSITS

The older Alluvial and Bay deposits identified on the cross sections along the east-west portions of the various alternatives are very similar to the lower sands with clay lenses. They consist of very dense sands interbedded with very stiff clays. The sands vary considerably in gradation, from fine relatively clean sands, to silty sands and clayey sands. The strength parameters and other soil parameters recommended for the lower sands with clay lenses may be used for these strata as well.

6.8 BEDROCK

The bedrock consists of interbedded Shales, Sandstones, sheared Shale Melange, and occasional Siltstone. The depth to rock varies from as little as one to two feet to over 200 feet. Contours of the elevations of the top of rock are shown on Plate 6-15.

The rock varies considerably from very weak highly sheared and weathered shales, and very low to medium strong sandstone. Unconfined compressive strengths of relatively intact specimens of shale range from 180 to 5,800 psi. Unconfined compression tests on intact specimens of sandstone indicate strengths ranging from as low as 1,000 psi up to 27,000 psi. The samples that had the very low strengths had failed along discontinuities in the rock specimens.

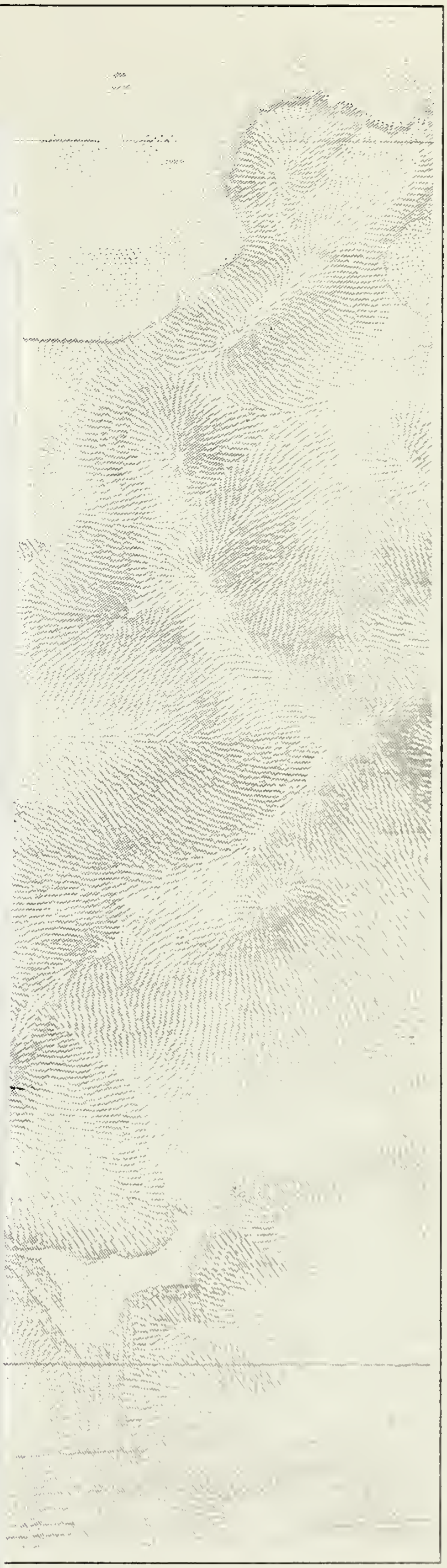
A number of point load index tests was also performed on intact samples of shale and sandstone. There is considerable variation in the results, from as low as 16 psi to 638 psi for sandstone; and from 6 psi to 213 psi for shale. The results of the point load index tests were correlated with fracture spacing, within each segment of the core from which the test samples were selected. Plate 6-16 shows the results of the tests plotted on a chart that includes guidelines for excavatability of rock (Navy DM-7 Manual). The results of the tests indicate that the majority of the rock can be excavated either by digging or by ripping. However, it should be expected that zones of fairly massive rock would be encountered that may require blasting to loosen the rock.

It must be emphasized that the majority of the rock recovered from the boreholes was highly sheared with considerable degree of weathering. However, there are zones within the weathered, sheared and fractured rock that are fairly intact. Based on the information from the boreholes drilled for this study, it is estimated (for the preliminary purposes of this study) that the zones of intact rock represent 10% to 15% of the rock cored in the borings. However, significant additional coring would be required to reliably estimate the percentage of sound rock that may be encountered along each of the alignments.

6.9 GROUNDWATER CONDITIONS

The groundwater table elevations measured in the boreholes drilled for this investigation varied between -3 feet and -10 feet (City and County of San Francisco Datum) in the low lying areas. In the high areas, on Rincon Hill, groundwater levels vary substantially with measurements in the range of +35 to +45 feet. It would appear that the high groundwater levels in the high ground area represent perched water tables above the Shale and Siltstone layers that may be the result of leakage from underground utilities.

In the area along the Embarcadero roadway, because of the proximity to the Bay, the groundwater levels should be on the average about 10 feet below ground level and should fluctuate with the tides.



Foldout

MAP OF ORIGINAL SHORELINE AND MARSH OUTLINE; U.S. COAST SURVEY MAP, 1856

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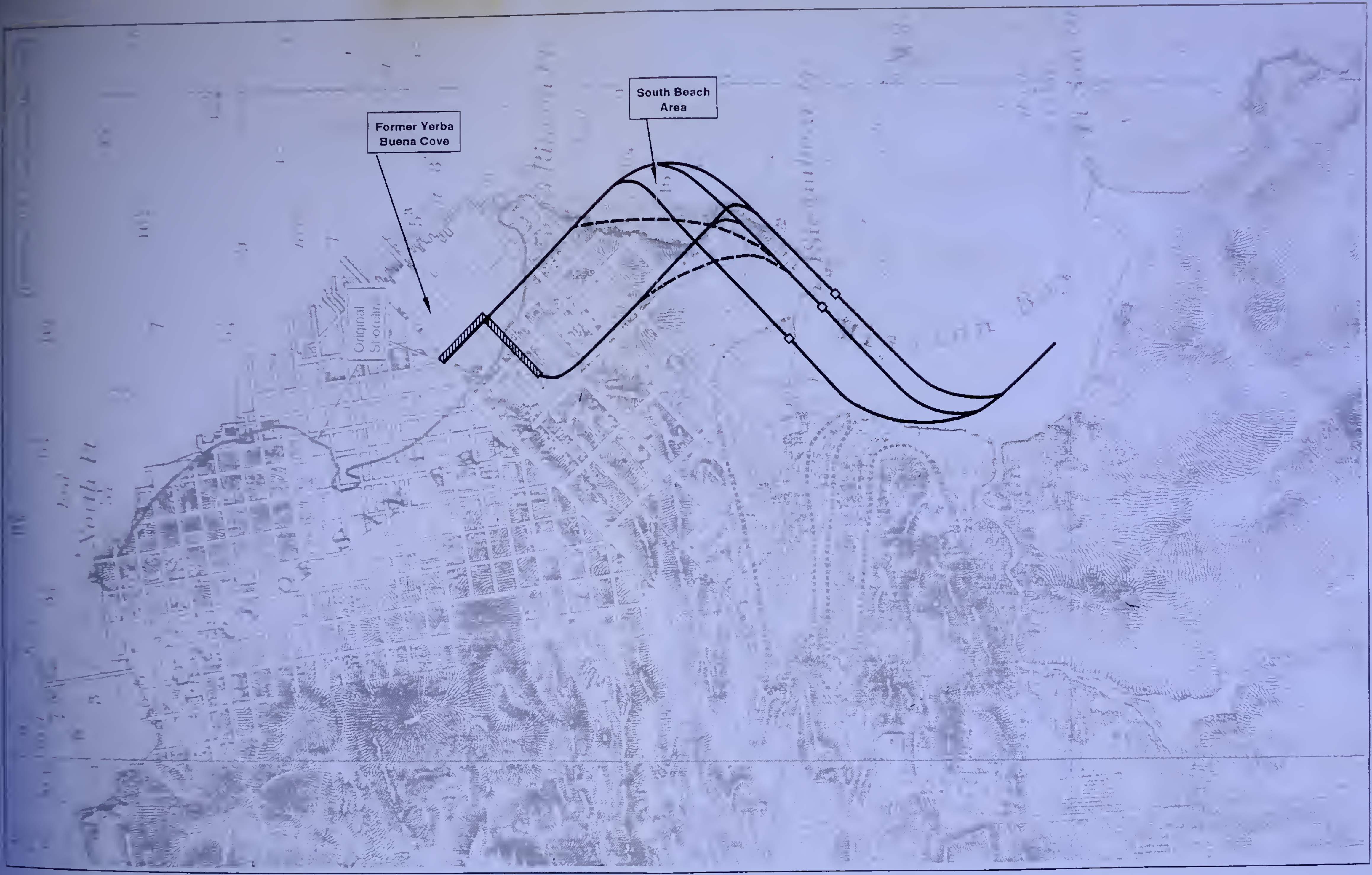
PLATE 6-1



MAP OF ORIGINAL SHORELINE AND MARSH OUTLINE; U.S. COAST SURVEY MAP, 1856

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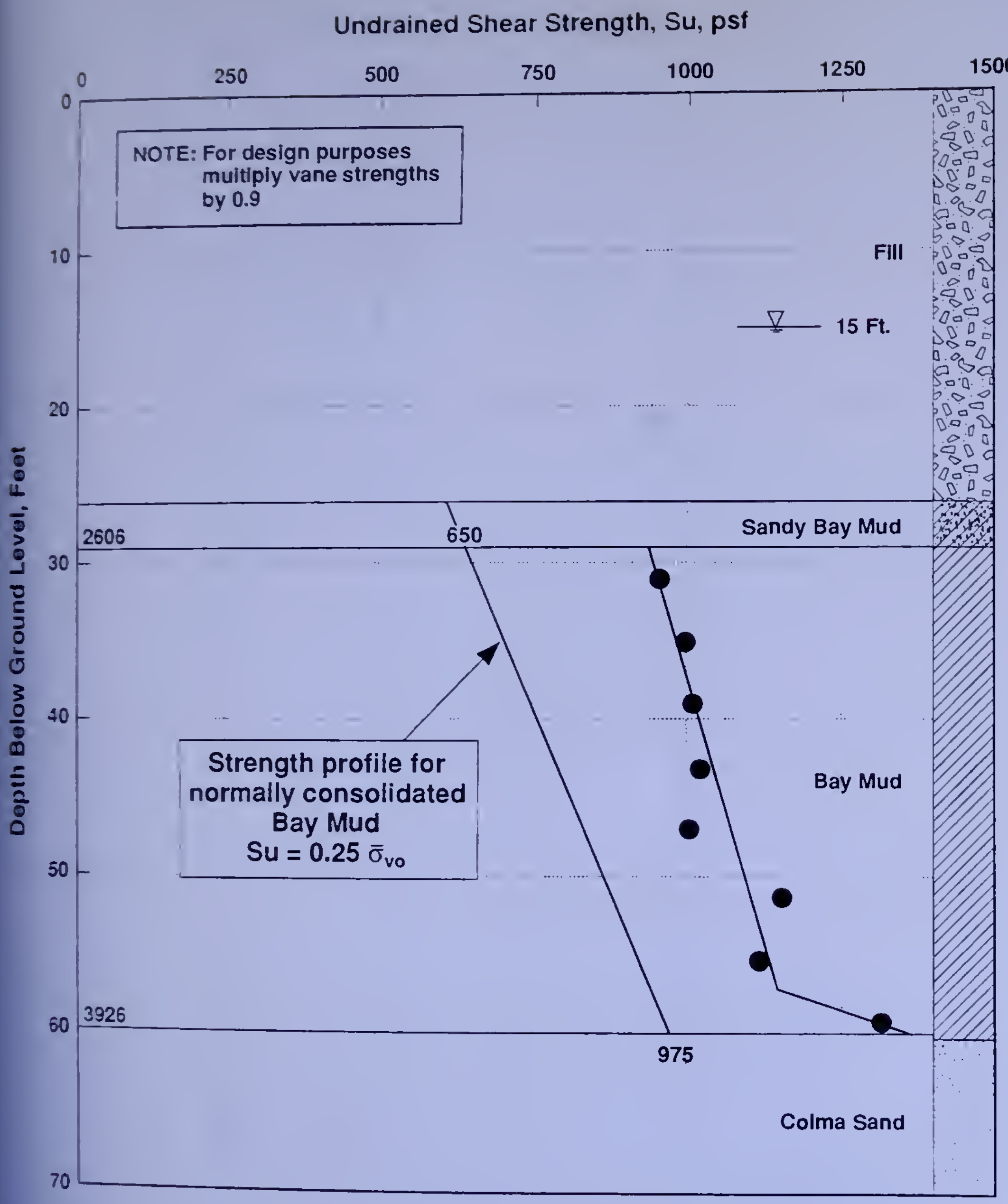
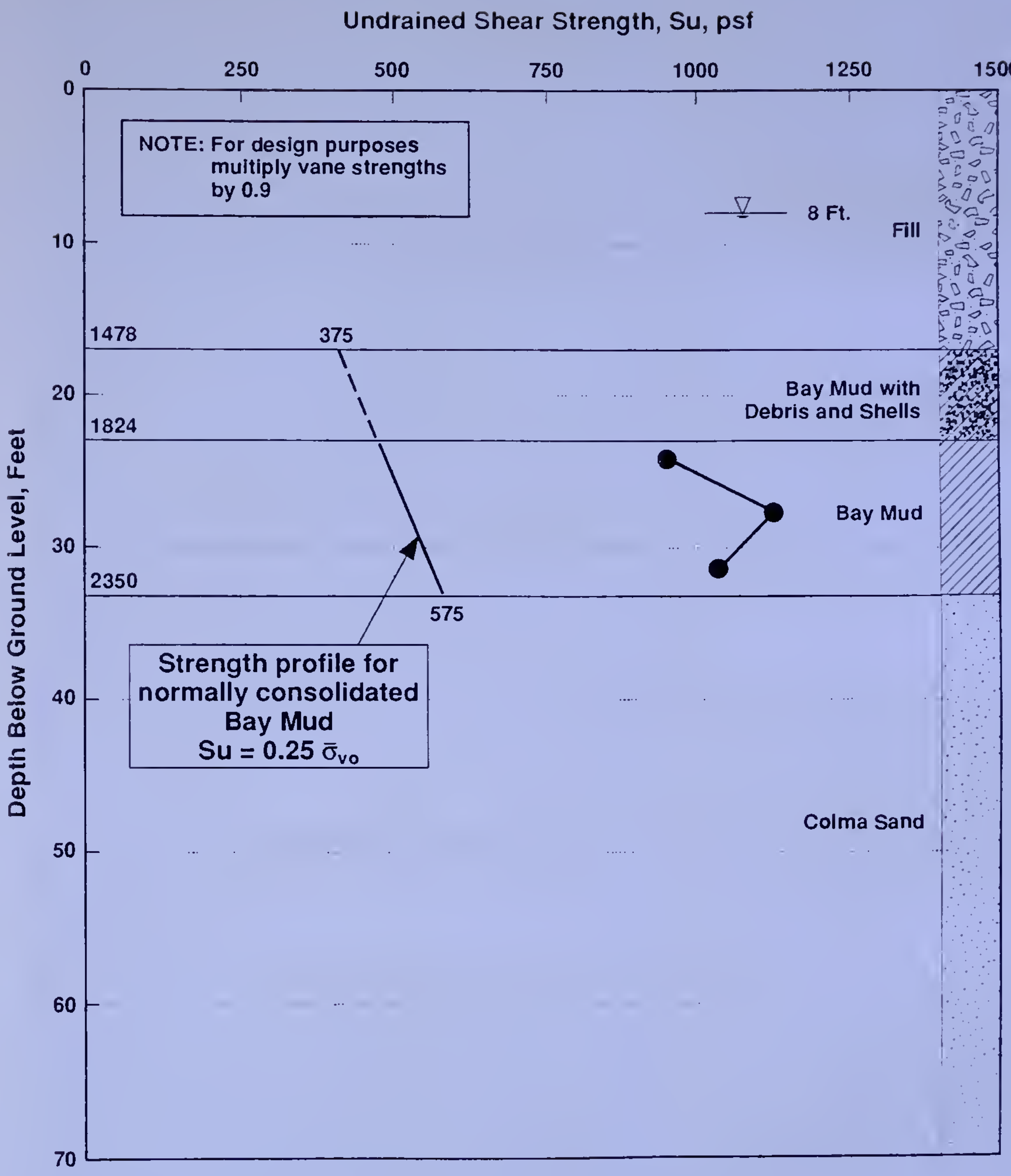
PLATE 6-1



(a) FIELD VANE: FV-3
1/2 BLOCK NORTH OF MISSION STREET



(b) FIELD VANE: FV-2
30 FT. SOUTH OF HOWARD STREET

RESULTS OF FV-2 AND FV-3 FIELD VANE TESTS

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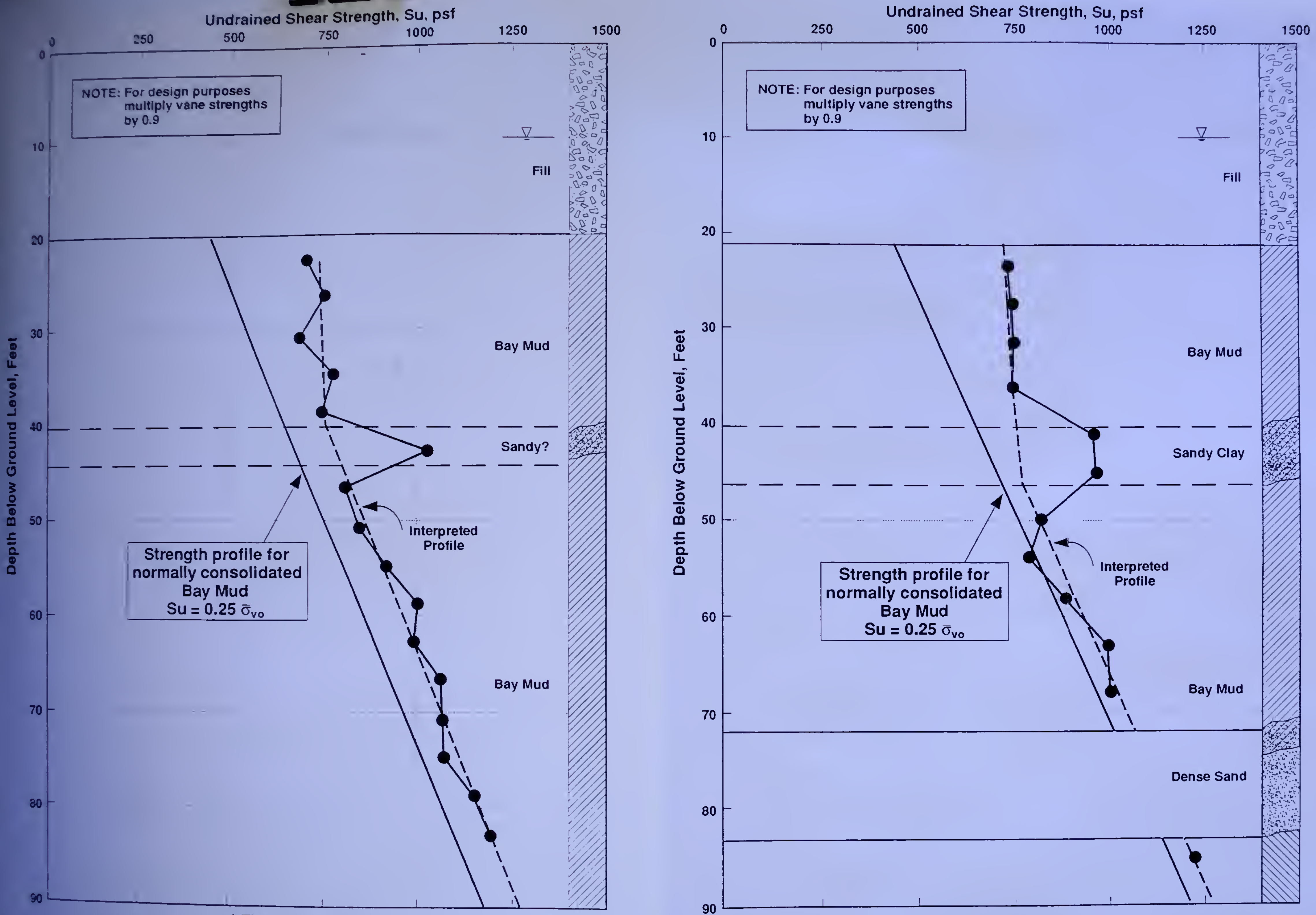

a) FIELD VANE FV-4
5TH AND TOWNSEND STS.

b) FIELD VANE FV-5
6TH AND TOWNSEND STS.

RESULTS OF FV-4 AND FV-5 FIELD VANE TESTS

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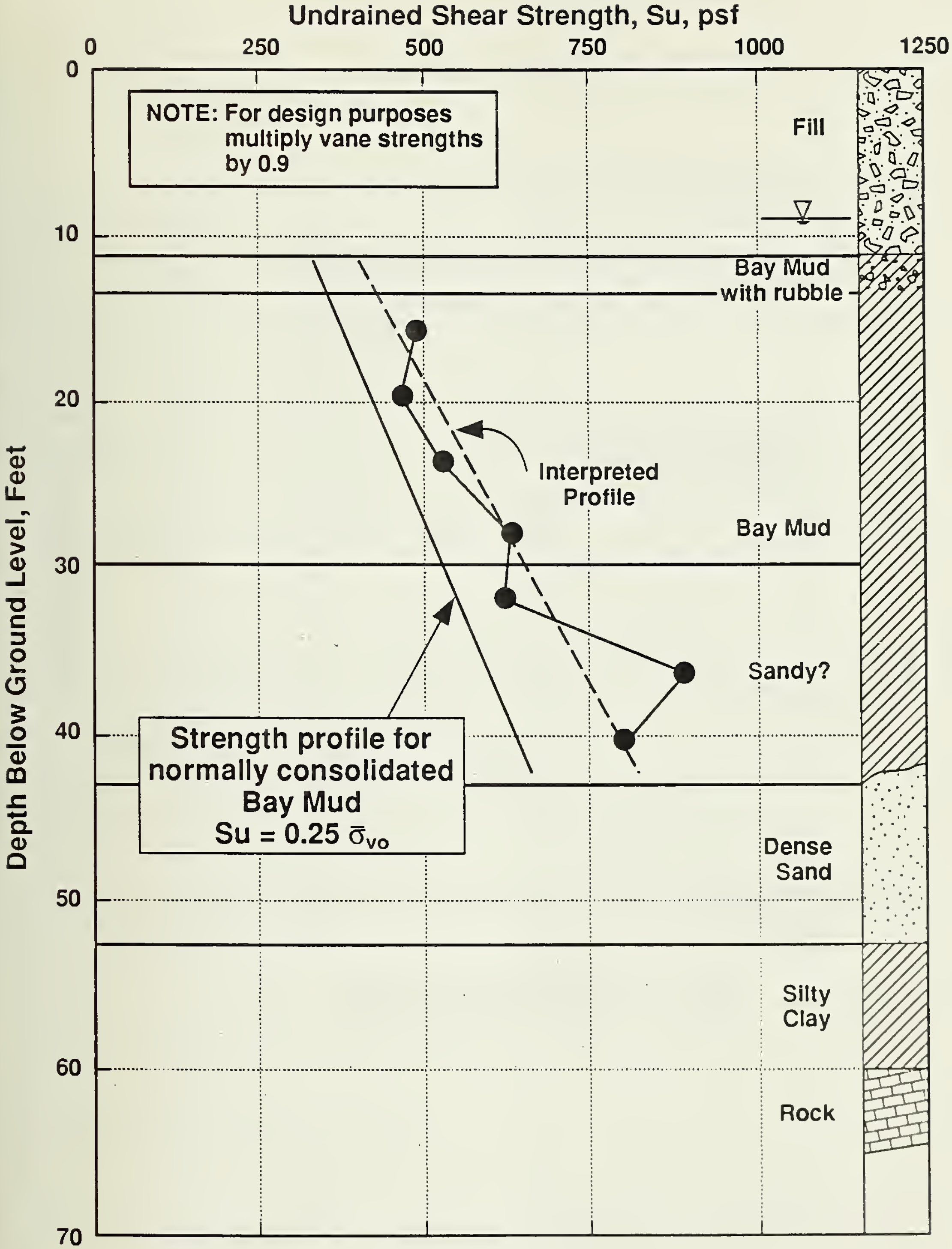
PLATE 6-13



FIELD VANE FV-6
ON BERRY ST. BETWEEN 7TH AND BERRY STS.

RESULTS OF FV-6 FIELD VANE TESTS

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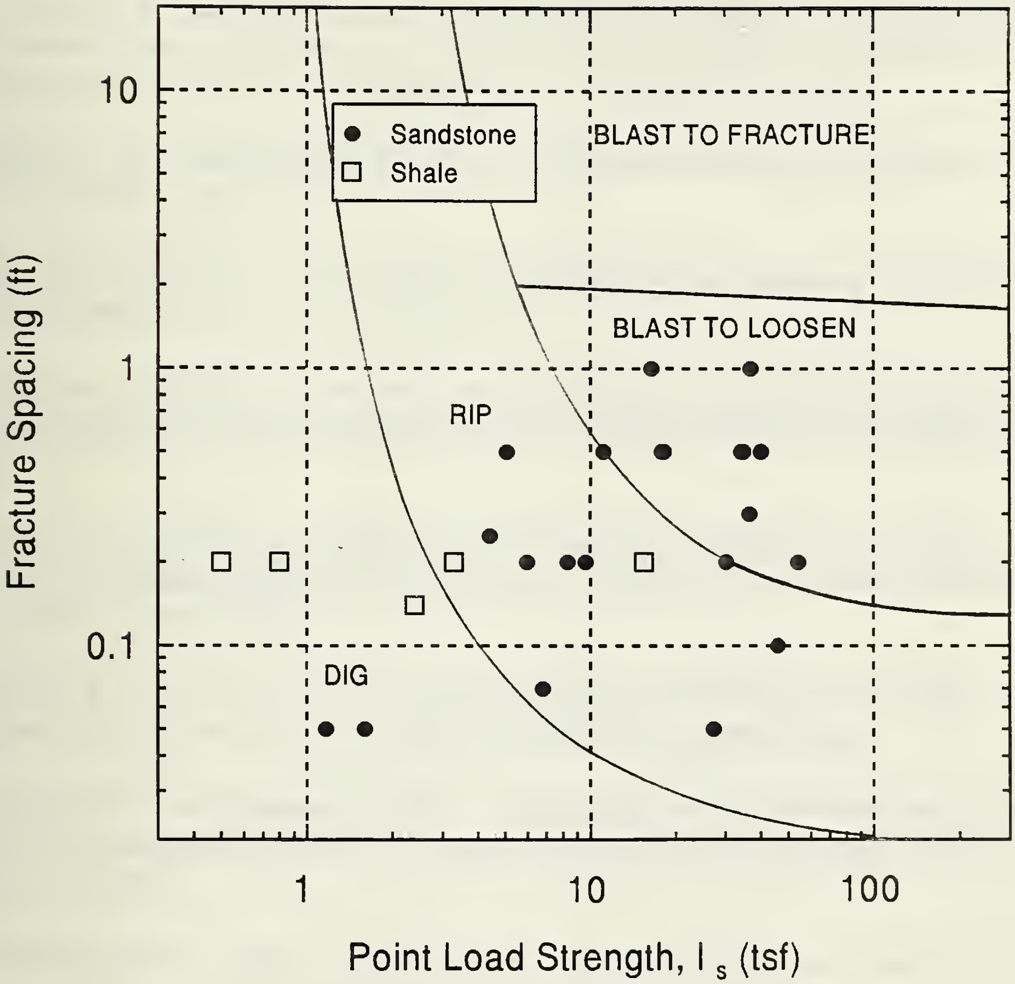
DAMES & MOORE PLATE 6-14



EXCAVATABILITY OF ROCK

September 1995
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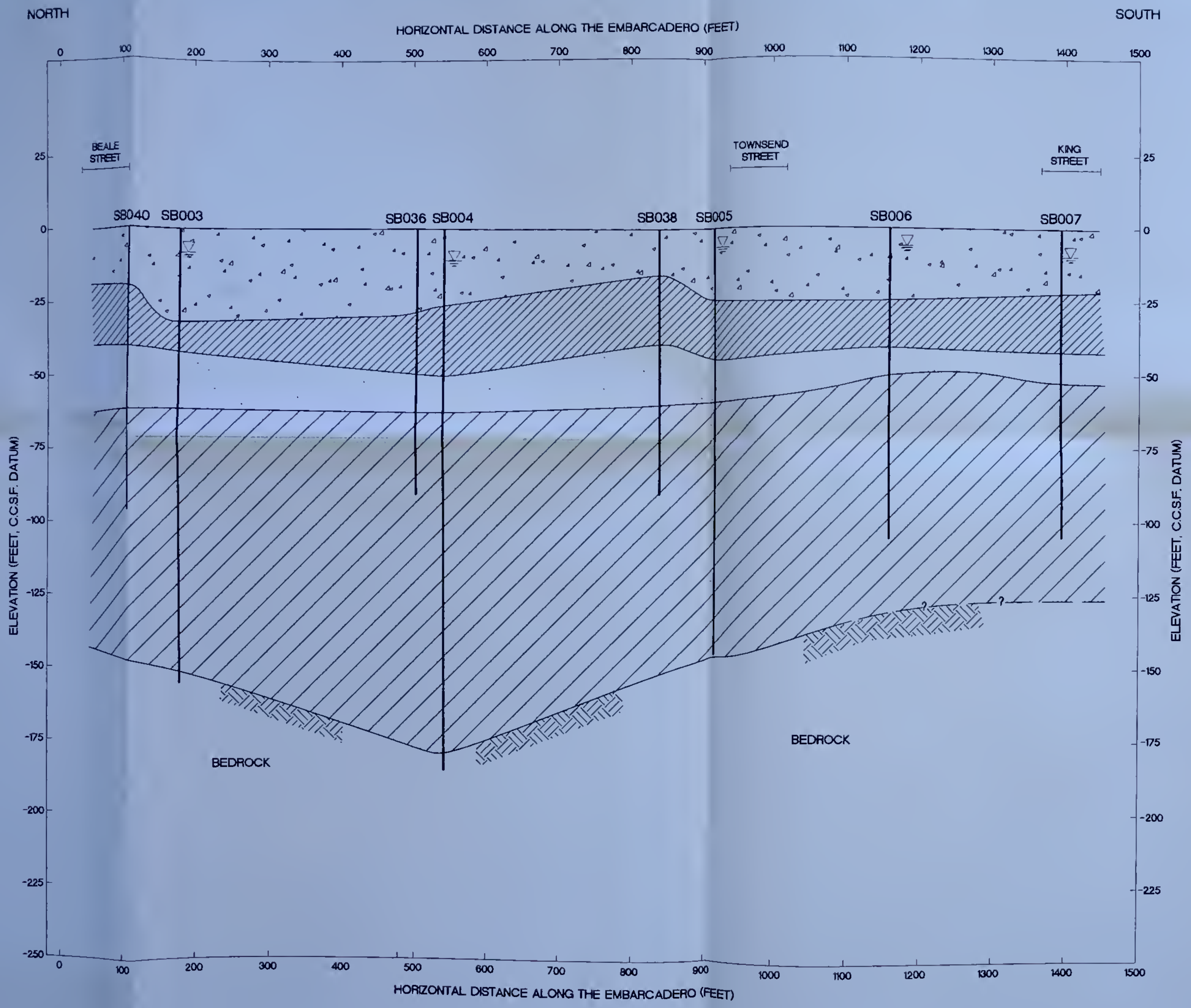
PLOT PLAN SHOWING EXISTING BOREHOLES
AND PROPOSED ALIGNMENTS

CalTrain San Francisco Downtown Station

DAMES & MOORE
SAN FRANCISCO, CALIFORNIA

Plate 5-1



LEGEND

- ⁘ FILL
- ▨ BAY MUD
- ☐ UPPER SANDS WITH CLAY LENSES
- ▨ OLDER BAY DEPOSITS: UNTERBEDDED DENSE SANDS AND STIFF CLAYS

NOTES:

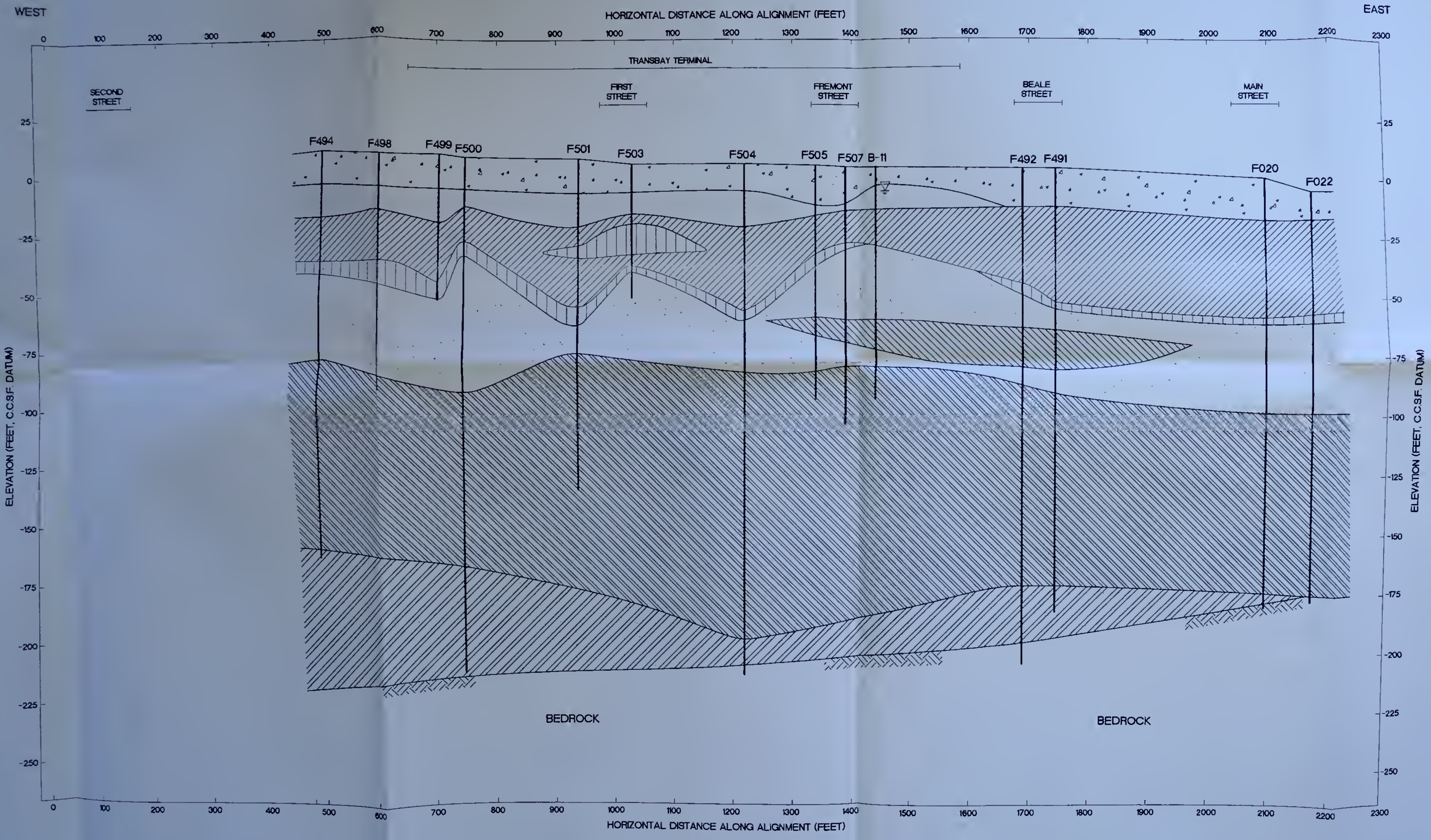
1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECION BORINGS HAVE BEEN PROJECTED PERPENDICUALR TO THE SECTION.
2. FOR DETAILED SOIL INFORMATION SEE BORING LOGS IN APPENDIX A.
3. FOR DETAILED DESCRIPTIONS OF THE MAJOR STRATA SEE THE MAIN TEXT OF THE REPORT.
4. GROUND SURFACE ELEVATIONS ARE BASED ON TOPOGRAPHIC MAPS PROVIDED BY AGS INC.

SUBSURFACE STRATIGRAPHY ALONG THE EMBARCADERO FROM BEALE TO KING STREET

CalTrain San Francisco Downtown Station

DAMES & MOORE
SAN FRANCISCO, CALIFORNIA

Plate 6-6



LEGEND

- ▫ FILL
- ▨ BAY MUD
- ☐ UPPER SAND WITH CLAY LENSES
- ▧ OLD BAY CLAY
- ▨ LOWER SANDS WITH WITH CLAYS LENSES
- ☐ DUNE SANDS
- ▧ MEDIUM STIFF SILTY CLAY
- ▥ LOOSE TO MEDIUM DENSE SILTY TO CLAYEY SAND (MARINE SAND)

NOTES:

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECION BORINGS HAVE BEEN PROJECTED PERPENDICUALR TO THE SECTION.
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SUBSURFACE STRATIGRAPHY
TRANSBAY TERMINAL

CalTrain San Francisco Downtown Station

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Plate 6-11